ONTARIO WATER RESOURCES COMMISSION

NOTES

of the

SENIOR

SEWAGE WORKS OPERATORS COURSE

April 22nd to 26th, 1963

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SENIOR SEWAGE WORKS OPERATORS COURSE

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SENIOR SEWAGE WORKS OPERATORS' COURSE

ONTARIO WATER RESOURCES COMMISSION

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SEWAGE WORKS OPERATORS' COURSE

ONTARIO WATER RESOURCES COMMISSION

April 22nd to 26th, 1963

Senior Course

Introductory Statement by D. S. Caverly, General Manager, OWRC

This Senior Course for sewage works operators completes the regular period of training prescribed by the Commission for those involved in the operation of sewage treatment works.

As agreed when these courses were undertaken, three periods of one week each were required in order to complete the instruction considered necessary for a candidate to write for the Certificate of Qualification. These lecture notes are printed so that each candidate may use them for reference purposes and also to continue his studies.

It is believed that this course of instruction, followed by the issue of a certificate, is a most desirable one to promote efficiency in the operation of sewage works systems, as well as stimulating the interest of those taking the courses. There is a very great responsibility placed on those in charge of the operation of sewage works systems. This course will assist them in carrying out these obligations. It is also important that those who have completed the courses continue their studies and be ever interested in becoming familiar with newer developments and techniques as well as keeping up to date at all times.

This issue of the Certificate of Qualification for the first time in Ontario is a forward step. We expect that those who receive these certificates will be worthy representatives of this Commission and those who aided in their instruction.

MATHEMATICS II

by

L. South

District Engineer - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 22, 1963

MATHEMATICS II

by

L. SOUTH

District Engineer

INTRODUCTION

This lecture is intended to review the basic mathematical rules discussed in the Intermediate Course and to present more typical problems and calculations.

ERRORS IN PREVIOUS COURSE

Since the printing of the previous lecture, i.e., Mathematical Problems in Sewage Treatment and Disposal in the Intermediate Course the following errors have been noted:

- P. 4 Example 3, last line should be 3.43 x $\underline{0.05}$

- P. 6 Example 2, third line should be 1 ml. titrated into a 100 ml. sample is equal to $\underline{2.0}$ ppm.

- P.11 Example 1, eleventh line should be,
  Vol. of wet sludge $= 60{,}000 \text{ lb.} \times \frac{1 \text{ gal.}}{10 \text{ lb.}} = 6000 \text{ gal.}$

$$\text{Time of pumping} = \frac{6000 \text{ gal.}}{100 \text{ gal/min.}} = 60 \text{ min.}$$

REVIEW OF RULES

Fractions - 2/6 means 2 divided by 6

- add and subtract fractions of like denominators,

  i.e., 2/6 + 1/3

  = 2/6 + 2/6 = 4/6 or 2/3

- multiply numerators by numerators and denominators by denominators 2/3 x 3/7 = 6/21

- it is understood that a whole number has a denominator of 1

$2 \times 1/4 = 2/1 \times 1/4 = 2/4 = 1/2$

- in division, invert the divisor and multiply

$3/2 \div 1/3 = 3/2 \times 3/1 = 9/2 = 4\ 1/2$

DECIMALS

- adding and subtracting decimals the numbers must be kept in their correct position to the right or left of the decimal point

$$\begin{array}{r} 3.00014 \\ +\ \underline{43.12012} \\ 46.12026 \end{array} \qquad \text{or} \qquad \begin{array}{r} 5.3157 \\ -\ \underline{2.1242} \\ 3.1915 \end{array}$$

- in multiplying add the total number of digits to the right of the decimal points and insert the same number in the answer

```
  4.01)  total of four digits
  5.23)  to right of decimal point
  1203
  802
2005
20.9723  total of four digits to right
         of decimal point
```

- in dividing the divisor must be multiplied by some multiple of ten to change it to a whole number. In order to maintain the equilibrium the dividend must be multiplied by the same multiple of ten.

```
      5.46  ÷  2.352

or    5460. ÷  2352.      (multiply both sides)
                                by 1000
               2.321
      2352 ⟌ 5460.000
             4704
              7560
             7056
               5040
               4704
                3360
```

- note any number of zeros can be added to the right of the last significant figure in the decimal portion.

Converting Fractions to Decimals and Decimals to Fractions

- Fractions can be converted to decimals by dividing the numerator by the denominator. Thus, $1/2 = 0.5$

or

$$\begin{array}{r|l} & 0.5 \\ \hline 2 & 1.0 \\ & \underline{1\,0} \\ & 0 \end{array}$$

The decimal point of the quotient is put directly above the decimal point of the dividend.

or convert $3/8$ to a decimal

$$\begin{array}{r|l} & .375 \\ \hline 8 & 3.000 \\ & \underline{2\,4} \\ & \,60 \\ & \,\underline{56} \\ & \,40 \\ & \,\underline{40} \\ & \,0 \end{array}$$

- In the above two examples the decimals can be written as their equivalent fractions:

$$0.5 = 5/10 = 1/2$$

$$0.375 = \frac{375}{1000} = 3/8$$

POWERS AND ROOTS

(Powers) - When a number is to be multiplied by itself we use a method of short hand:

$2 \times 2 = 2$ squared or $2^2 = 4$

Powers of 2	Powers of 3
$2^1 = 2$	$3^1 = 3$
$2^2 = 4$	$3^2 = 9$
$2^3 = 8$	$3^3 = 27$
$2^4 = 16$	$3^4 = 81$

Powers of 10

$10^1 = 10$

$10^2 = 100$

$10^3 = 1000$

$10^4 = 10,000$

$10^5 = 100,000$

$10^6 = 1,000,000$

Roots) - The square root of a number is that number which when multiplied by itself will give the number. Thus the square root of 4 noted as $\sqrt{4}$ is equal to 2

$\sqrt{1} = 1$

$\sqrt{4} = 2$

$\sqrt{9} = 3$

$\sqrt{16} = 4$

The cube root, fourth root and such are seldom used but should be noted, thus the cube root of 8 noted as

$\sqrt[3]{8} = 2$

the fourth root of 16 noted as $\sqrt[4]{16} = 2$

ONVERSION FACTORS

1 acre = 43560 square feet

1 cubic foot = 6.25 gallons

1 cubic foot per second = 540,000 gallons daily

1 million gallons per day = 1.85 cubic feet per second

degrees centigrade = $9/5 \times {}^{o}C + 32$ = degrees fahrenheit

degrees fahrenheit = $5/9 \times {}^{o}F - 32$ = degrees centigrade

1 foot of water = 0.43 pounds per square inch

1 pound per square inch = 2.3 feet of water

1 gallon water = 10 pounds

1 grain per gallon = 14.3 parts per million

1 gram = 0.00205 pounds

1 gram/liter = 1000 parts per million

1 milligram/liter = 1 part per million

SAMPLE CALCULATIONS

Example 1

The sludge age is the calculation of the length of time a unit of activated sludge remains in the sewage treatment plant. The recommended sludge age for most plants is 3 1/2 days. Sludge age can be calculated by using the following formula:

$$\text{Sludge age (days)} = \frac{V \times A}{Q \times C}$$

V is aerator volume in gallons

A is average aerator concentration of suspended solids in parts per million

Q is the sewage flow in gallons/day

C is the primary settling tank effluent suspended solids in parts per million

Question: What is the sludge age with the following conditions: sewage flow 1,500,000 gallons daily, suspended solids in raw sewage = 200 ppm. aeration tank concentration of suspended solids = 2400 ppm. primary settling tank removal efficiency = 60% volume of aeration tank is 40,000 cu.ft.

Solution:

$$V = 40{,}000 \text{ cu.ft.} \times \frac{6.25 \text{ gal.}}{\text{cu.ft.}} = 250{,}000 \text{ gal.}$$

$$A = 2400 \text{ ppm.}$$

$$Q = 1{,}500{,}000 \text{ gal.}$$

$$C = 200 \text{ ppm.} - 200 \text{ ppm.} \times \frac{60}{100} = 80 \text{ ppm.}$$

$$\text{Sludge age} = \frac{250{,}000 \text{ gal} \times 2400 \text{ ppm}}{1{,}500{,}000 \frac{\text{gal}}{\text{day}} \times 80 \text{ ppm}} = 5 \text{ days}$$

Example 2

The percent sludge being returned can be calculated by knowing the suspended solids concentrations in parts per million for, the aeration tank, the return sludge and the primary settling tank effluent by using the following formula:

$$\text{Percent return sludge} = \frac{Ca - Cp}{Cr - Ca} \times 100$$

Ca is suspended solids concentration in aeration

Cr is suspended solids concentration in return sludge

Cp is suspended solids concentration in primary effluent

Question: What is the percent return sludge in a plant where the suspended solids concentrations are as follows:

Aeration tank 2200 ppm.

Return sludge 1200 ppm.

Primary settling tank effluent 100 ppm.

$$\% \text{ return sludge} = \frac{2200 \text{ ppm.} - 100 \text{ ppm.}}{12000 \text{ ppm.} - 2200 \text{ ppm.}} \times 100\% = 21.5\%$$

Example 3

The following pertinent data is noted for a typical activated sludge plant.

2-primary settling tanks each 52' x 16' x 10' W.D. with four full width weirs at the effluent end.

4-aeration tanks each 72' x 18' x 10' W.D. air supply

2-blowers each at 850 ft^3/min.

2-final settling tanks each 56' x 16' x 10' W.D.

1 chlorine contact tank 37' x 12' x 7' W.D.

1 Digester 52' ø x 24' W.D.

The results of flow studies and samples indicate:

flow 1,000,000 gallons daily

raw B.O.D. 200 ppm.

primary B.O.D. 130 ppm.

final B.O.D. 15 ppm.

raw Suspended Solids 250 ppm.

primary Suspended Solids 100 ppm.

final Suspended Solids 15 ppm.

raw sludge % solids 3%

raw sludge % volatile 70%

digested sludge % solids 10%

Determine the following:

(1) Detention time in primary tanks

(2) Weir overflow rate in gal./ft./day in primary

(3) Air supply in ft^3/gal. sewage

(4) Detention time in aeration tank with 25% return

(5) Percent removal of B.O.D.

(6) Percent removal of Suspended Solids

(7) Detention time in chlorine contact tank

(8) Pounds of B.O.D. removed

(9) Pounds of Suspended Solids removed

(10) Gallons of sludge pumped

(11) Pounds of volatile solids to digester

(12) Cubic feet of digester space per pound of volatile solids added

(13) Gallons of digested sludge to be removed each day

Answer:

$$\text{(Primary) Volume} = 2(52' \times 16' \times 10')$$

$$= 16{,}700 \text{ ft}^3 \times \frac{6.25 \text{ gal.}}{\text{ft}^3}$$

$$= 104{,}000 \text{ gal.}$$

1) Detention time = $\dfrac{104,000 \text{ gal.}}{\dfrac{1,000,000 \text{ gal.}}{24 \text{ hr.}}}$ = 0.104 x 24 hr. = 2.5 hr.

2) Length of weirs 2(16' x 4) = 128'

Overflow rate = $\dfrac{1000,000 \text{ gal/day}}{128 \text{ ft.}}$ = 7,800 gal/ft/day

3) Volume of air = $2(850 \text{ ft.}^3/\text{min.} \times \dfrac{60 \text{ min.} \times 24 \text{ hr.}}{\text{hr.} \quad \text{day}})$

= 2,450,000 ft.^{3}/day

Air supply = $\dfrac{2,450,000 \text{ ft.}^3/\text{day}}{1000,000 \text{ gal./day}}$ = 2.45 ft.^{3}/gal.

4) Volume of aeration tanks 4(72' x 18' x 10')

= 5200 ft.^{3} x $\dfrac{6.25 \text{ gal.}}{\text{ft.}^3}$ = 324,000 gal.

Detention time = $\dfrac{324,000 \text{ gal.}}{1,250,000 \text{ gal./24 hr.}}$ = 0.26 x 24 hr.

= 6.2 hr.

5) % B.O.D. removed = $\dfrac{200 - 15}{200}$ x 100 = 92.5%

6) % Suspended Solids removed = $\dfrac{250 - 15}{250}$ x 100 = 94.0%

7) Volume chlorine contact tank = 37' x 12' x 7'

= 3100 ft.^{3} x $\dfrac{6.25 \text{ gal.}}{\text{ft.}^3}$

= 19,500 gal.

Detention time = $\dfrac{19,500 \text{ gal.}}{1,000,000 \text{ gal./24 hr.}}$ = .0195 x 24 hr.

= 0.47 hr.

= 0.47 hr. x $\dfrac{60 \text{ min.}}{\text{hr.}}$ = 28.2 min.

8) Pounds of B.O.D. arriving = 1000,000 $\dfrac{\text{gal.}}{\text{day}}$ x $\dfrac{10 \text{ lb.}}{\text{gal.}}$ x $\dfrac{200 \text{ lb.}}{1,000,000 \text{ lb.}}$

= 2,000 lb./day

Pounds of B.O.D. leaving $= 1000{,}000 \ \frac{\text{gal.}}{\text{day}} \times \frac{10 \text{ lb.}}{\text{gal.}} \times \frac{15 \text{ lb.}}{1{,}000{,}000 \text{ lb.}}$

$= 150$ lb./day

Pounds of B.O.D. removed $= 2000 \ \frac{\text{lb.}}{\text{day}} - \frac{150 \text{ lb./day}}{\text{day}} = 1850$ lb./day

(9) Pounds of Suspended Solids arriving =

$$1000{,}000 \ \frac{\text{gal.}}{\text{day}} \times \frac{10 \text{ lb.}}{\text{gal}} \times \frac{250 \text{ lb.}}{1000{,}000 \text{ lb.}} = 2500 \text{ lb./day}$$

Pounds of Suspended Solids leaving =

$$1000{,}000 \ \frac{\text{gal.}}{\text{day}} \times \frac{10 \text{ lb.}}{\text{gal.}} \times \frac{15 \text{ lb.}}{1000{,}000 \text{ lb.}} = 150 \text{ lb./day}$$

Pounds of Suspended Solids removed =

2500 lb./day - 150 lb./day = 2350 lb./day

(10) 2350 lb./day of suspended solids removed

raw sludge is 3% suspended solids

$\therefore$ 3% of raw sludge = 2350 lb.

100% of raw sludge $= \frac{2350}{3} \times 100 = 78{,}333$ lb. $= 7{,}833$ gal.

$\therefore$ amount of raw sludge pumped to digester = 7,833 gallons

(11) Raw sludge is 70% volatile

$\therefore$ $2350 \times \frac{70}{100} = 1645$ lb. of volatile solids to digester

(12) Volume of digester $= \pi \times \frac{52^2}{4} \times 24 = 51{,}000 \text{ ft.}^3$

Digester loading $= \frac{51{,}000 \text{ ft.}^3}{1645} = 31 \text{ ft.}^3$/lb. volatile solids added/day

(13) 7833 gallons of raw sludge containing 3% dry solids is pumped to the digester each day.

Volume occupied by dry solids $= \frac{3}{100} \times 7833 = 235$ gal.

The same volume will be occupied by the dry solids whether the moisture is increased or decreased.

Digested sludge contains 10% dry solids

10% digested dry solids occupy 235 gal.

1% digested dry solids occupy 23.5 gal.

100% digested dry solids occupy 2350 gal.

∴ an average of 2350 gallons/day of digested sludge will have to be pumped from the digester.

ORGANIZATION AND MANAGEMENT OF SEWAGE PLANT STAFF

by

B. C. Palmer

Director, Plant Operations - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 22, 1963

ORGANIZATION AND MANAGEMENT OF SEWAGE PLANT STAFF

by

B. C. PALMER

Director, Plant Operations

INTRODUCTION

In addition to learning the techniques and skills of handling pumps, motors, scrapers, heat exchangers, blowers, test tubes, flasks and the many other mechanical, chemical and biological components of your plant, you have manpower resources to manage. This lecture suggests ways to do this.

THE CHALLENGE

Men are not machines. But like machines, they perform work. Your chief challenge lies in drawing out the maximum potential of your manpower. Whether you have 70 men or only yourself to supervise, your problem is basically one of maximizing results. We all understand the advantages of proper maintenance of an automobile engine. The best maintained engines give the most miles to the gallon. Men are like that too. The difference is that an engine has only a few dozen separate parts, all of which a skilled mechanic can control and understand. In this age of careful quality control, we learn to depend on uniformly good service from our tools and machines. The laws governing motor efficiencies, metal stress, electrical control, chemical reactions are, to great extent, known and can be learned by most of us. There are laws governing human behaviour, individual differences and similarities in efficiency, the effects of emotional stress, nervous system reactions, physical health, and past experience but these laws are less well known.

Our degree of success in managing both ourselves and others depends primarily on our ability to understand people, especially the forces and influences that make people do what they do. This task appears to be so great, and we have so many personal experiences that indicate our own inability to understand others, that we are sometimes tempted to conclude that understanding people is like trying to understand the path of a tornado. We never will, so why bother to try? Let us develop skill in handling others. A fundamental skill in understanding and handling people is "empathy", the ability to put ourselves in the other person's place and looking at a problem from his point of view. This does not mean that we always accept his point of view as being right. It means that we recognize how he feels about it.

RGANIZATIONAL SKILL

Whenever two or more people work together in a common ause, some organizational structure takes place. For instance, ı the home the man earns the money and the woman spends it. ıis is a very highly developed organizational skill, especially ith one of the partners. Even if yours is only a one man plant, ɔu still have a number of "organizing" problems. For example:- ı small plants, only one man is required, full or part time. e does everything. Where the size of the plant, its physical ayout and the flow characteristics are such that one man cannot andle all the work, more men are needed. Areas of responsibility ıst be laid down in order to avoid confusion and conflict. Two ajor divisions of work allocation are by,

(a) Responsibility

(b) Shift

(a) Responsibility

In delegating work to others, it is helpful to write ɔwn groups of responsibilities that are related. These may ɔrm basic patterns around which to build an organization chart. ıch a group listing might look like this:

Process Control

(a) Laboratory

(b) Plant

Repair and Maintenance

(a) Electrical

(b) Mechanical

(c) Buildings and Grounds

Process Operation

(a) Primary clarifiers

(b) Biological section

(c) Sludge conditioning

(d) Sludge disposal

Administration

(a) Overall supervision

(b) Training

(i) Process
(ii) Equipment
(iii) Safety

(c) Records and reports

(d) Public relations

(e) Purchasing

(f) Plant security

All of these responsibilities may exist in a one man plant, in which case, of course, that man is responsible for all of them. In a very large plant it may be necessary to assign several men to each individual job listed above.

How to Determine the Proper Number of Staff

This is one of the most difficult problems facing administrators. It is evident that the minimum number of staff to effectively do the work is the goal to strive for. Factors to be considered are:

1. Type of treatment

2. Volume of flow

3. Pattern of flow

4. Quality of wastes

5. Plant layout including buildings and grounds

6. Age of plant and condition of buildings and equipment

7. Location with regard to other buildings, especially residences

8. Ramifications of mechanical failures at the plant

9. Security requirements

10. Number, location and type of satellite installations such as remote lift stations to be serviced.

11. Availability of other groups to handle such items as snow plowing and grass cutting.

12. Availability of qualified staff at the salary levels offered.

13. Feasibility of automatically signalling alarms to an action point.

14. Union agreements.

The problem of the actual number of staff is usually resolved initially by those holding the purse strings. However, after a plant is in operation, the superintendent is given a basic responsibility for the successful operation of the works so this puts him in an excellent position to influence decisions on future staff requirements.

Frequent review of the interrelationships of the factors just listed is essential if the operation is to avoid drifting into the evils of over or under staffing. With an almost universally increasing load coming to your plants, the possibilities of laying off staff are usually not good. However, there is a real challenge in holding the line on increasing staff as the work load increases due to increased flow or maintenance. When the pressure of a backlog of unfinished work starts to build up, the usual temptation is to start to agitate for more staff. Before you reach this point, you should first carefully go over all your work assignments to determine,

(a) If all the things that are being done really need doing

(b) If there is a better, quicker way of doing all the jobs

You, as a plant superintendent, are in a better position than anyone else to evaluate the work load. Your immediate superior is not, as a rule, close enough to the day to day activities to know as much as you do. Those under you are too close to the problem to always give you completely accurate information. Few will admit that they are underworked.

Getting the Most out of Yourself and Your Men

1. Allow yourself all the time it takes to properly organize your staff, train them and supervise them. There is nothing more important for you as a supervisor to do. If you have five or six men under you, but find yourself frequently on the end of a broom, wrench, hose or mop, you aren't able to properly organize the work of the others.

2. Keep in mind that your men are all human beings.

3. Acknowledge their strengths and encourage them by so doing.

4. Help them to overcome their weaknesses.

5. Give your men responsibility with the authority necessary to go with it. If you make a man responsible for the proper maintenance of a pump and then delegate someone else to look after it without consulting him, he will experience no real feeling of responsibility for its welfare. The gentle art of delegating responsibility without losing control is worthy of your best efforts.

6. Firmness and fairness are qualities understood and appreciated by all men. Your staff looks to you for stability and solidarity. They don't want a wishy washy boss who makes erratic decisions. Your men like to know where they stand with you.

7. Keep open channels of communication between your staff and yourself and between yourself and your boss. Give every suggestion from your staff sympathetic careful consideration. You won't kill a man's spirit by turning down a suggestion. You can, by making fun of his suggestions or being indifferent about them. Remember, you need him or he would't be there.

8. Practice the common courtesies. Let your men know you expect them to also. One of the greatest obstacles in our way is lack of consideration for others.

9. Keep out ahead of your staff. You are a leader and everyone expects you to act like one. Don't be like the scout master who came wheezing and puffing up a mountain path. He paused to wheeze out the following to some fishermen, "Have you seen a group of boys pass this way? I'm their leader."

Shift Schedules

10. Give a lot of thought to the preparation of shift schedules. Men can become very excited over supposed inequalities in a shift schedule. Don't let resentment over a shift schedule smoulder in a man too long. Before long, he will be putting in time but not doing too much. Make sure total hours balance out. Space out work loads. Work out arithmatic of hours available and staff required for 24 hour work.

11. Develop an 'esprit de corps'. Some plants take great pride in their work. They are convinced they have the best operation in the province. They usually do. Frequent staff training conferences are desirable. They impress the operators with the importance of the work they are doing. Occasional staff social events such as a Christmas Party or Picnic are desirable.

12. Pay Schedules

Superintendents have to balance between the management problem of cost control and the operator problem of wanting more pay. Since you are the "man in the middle" you have an important role to play. The safest short run policy is, of course, to do nothing about this and let the great white fathers in Toronto or City Hall determine the fate of you and your staff. This works providing the great white fathers are omniscent i.e. all wise and are acutely aware of all the problems. However, if there are apparent inequalities or injustices in pay rates, then you have an obligation to investigate and take action. Find out from your boss what salary policies are in effect. Armed with this information, you will then be in a position to act intelligently on behalf of your men.

Don't be just a go between from your men to your boss or vice versa. Your boss will expect you to have opinions on salary rates. If he asks, you should be prepared to offer some constructive suggestions. If your suggestions are always for raises no matter what, don't be surprised to find that he fails to ask for your opinion in the future. But he will appreciate intelligent, well thought out responsible suggestions.

Accident Prevention and Reporting

1. Set up reporting and records system.

2. Eliminate all the unnecessary hazards you can before accidents draw them painfully to your attention.

3. A plant superintendent is frequently the goat when interested parties such as the Chief Coroner start looking around for some place to send a criminal negligence charge.

4. Make sure your men know what to do in case of an accident. If you don't know, you can't expect them to.

Budget Control

Every plant operator has an obligation to help control expenditures. Go over the operating cost figures with your staff so they will understand the limits and objectives planned in the budget. The OWRC works on an annual budget that is drawn up in the fall. Find out your employer's budget policies and be prepared to point out to him work that should be included in the next budget. After budgets are set, it takes a revolution and three assassinations to change them.

In Summary

Remember, your chief responsibility as a plant superintendent is as an organizer, planner, administrator. It may be that if yours is a one man plant, you will have all the operator duties as well but, without planning, you can't control your own work or that of others. Use initiative and imagination. The challenge of operating a waste water treatment plant effectively and economically is exciting and has a lot of job satisfaction to most plant superintendents. If you accept this challenge, you and your staff will get a huge amount of job satisfaction.

SEWER CONSTRUCTION

by

W. A. S. Marshall

Division of Construction - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 22, 1963

SEWER CONSTRUCTION

by

W. A. MARSHALL

Construction Division

In discussing the subject of sewer construction, I intend to present some of the methods of construction and problems encountered in the work from the time the contractor moves his equipment on to the job until restoration is completed and accepted by the owner. The success of the work depends to a great extent on the ability of the superintendent to make proper use of the equipment and workmen engaged on the job.

Generally a municipality or company desiring to install sewers will have their engineering department or a firm of consulting engineers undertake the design and the preparation of plans and specifications. The plans should show as much detail as possible; that is location of roadways, curbs, sidewalks, service poles, trees, buildings, known underground services and the line of the proposed sewer. The profile, which is a vertical section along the line of the sewer should show the existing ground elevation, elevations of the proposed sewer, elevation of basements of buildings which are to be served by the sewers. Chainages are marked at 100 ft. stations and intermediate chainages are given for manholes, services and other structures to be constructed. The elevation datum and intermediate bench marks are given and identified.

The specification should be clearly set out to cover the work involved. Some specifications contain standard clauses which may not pertain to the particular job being undertaken. Such clauses tend to confuse the intent of the contract and should be omitted. The specifications in general set out the methods and materials, tests, method of measurement and payment.

Tenders for sewer work are generally set up on a unit price basis. Excavation and backfill including dewatering and shoring are paid for by the cubic yard; supply and installation of sewers by the lineal foot; manholes by the vertical foot or unit manhole; restoration of road surfaces may be included in the price for excavating and backfilling or may be set up on a square yard basis.

On Commission projects the contractor is required to do the field layout for line and grade using the information given on the plans and profiles and the given bench marks. A bench mark is a point of known elevation. The resident engineer is then responsible for checking the contractors work. In this way the chance of error is greatly reduced. The layout man will set a line of stakes offset 10 feet or so from the line of the proposed sewer at intervals of 50 ft. corresponding to the stations given on the plans. The stakes are planted to the side where trenches or excavated material will not interfere with them. Stakes will also be placed for manholes and services. These stakes are left with about a foot above ground so that the station and cut can be marked on the faces. A second stake is placed on line beside these stakes driven almost flush with the ground. The elevation of the top of the second stake is established by the use of an engineers level and rod relating the elevation to the given bench marks. The difference between the elevation of the top of this stake and the elevation of the sewer invert at that station is called the cut. Batter boards are then set at these stations which consist of a stake which extends above the ground about three to four feet with a horizontal cross piece set a full number of feet above the invert elevation of the sewer. A grade rod is made with a shoe on the lower end which will sit on the lower inside surface of the pipe and a horizontal cross piece is fixed at a height equal to the difference in elevation of the sewer invert and the top of the batter board. As each pipe is laid the grade rod is set on the invert of the sewer and the cross piece is lined up with three successive batter boards. If the cross piece does not line up, the pipe has to be lowered or raised as the case may be.

The line of the sewer is checked by suspending a plumb bob from a rod 2" x 2" held horizontally, with the distance of the off set marked. One end of the rod is held on the off set stake and the plumb bob is suspended over the pipe from the mark on the rod. The centre line of the pipe is lined up under the plumb bob. To do this between stations a string line is stretched between the stakes at successive stations and the off set is measured from the string. Once the layout is sufficiently advanced the construction can commence.

The contractor may have available several types of excavating machinery. Some of the more common types used are backhoes, clamshells, draglines, trenching machines, front end loaders and bull dozers. The backhoe has seen the most use in recent years for sewer construction. They are available in capacities varying from 1/2 to 3 or more cubic yards. They are convenient for widths exceeding three feet and depths up to 25 ft. With a hook in the back of the bucket it can be used with a sling to lower pipe into the trench.

The Clamshell

The clamshell is well suited where close sheeting and vertical lift are required. It may be used in conjunction with the backhoe where the depth of trench requires more than one lift.

Draglines

Draglines may be used in open country for a large part of the excavation where the trenches are allowed to take their natural slope. It may also be used in conjunction with the backhoe where two lifts are required to excavate the first lift and to remove the earth which the backhoe excavates on the second stage. The dragline is also able to cast the excavated material well back from the side of the trench which will reduce the danger of the banks slipping.

Trenching Machines

Trenching machines have been used for shallow trenches of moderate width in cohesive soils where only skeleton sheeting is required. If large boulders or unstable ground are encountered the trenching machine has to be taken off line and another excavating machine brought in. This way the contractor has two pieces of expensive machinery on the job with one sitting idle. It is this reason and the limited operating depth that this type of machine is not so popular in the sewer construction field. Front end loaders and bulldozers may be used for removing the first few feet where the depth of trench is slightly greater than the reach of a backhoe. It may also be used to level up the bottom of the trench where larger sized sewers are being installed. Both machines are used for backfilling. Some front end loaders have a side tip bucket which is very convenient where working space is confined.

As well as the major earth moving equipment suitable portable pumps, compressors, jackhammers and rock drilling equipment should be on the site for immediate use where wet soils, rock or pavement are encountered. The usual hand tools such as shovels, picks, sledge hammers are also necessary.

In areas where the roads are paved with hot mix asphalt or concrete these surfaces have to be cut along a line on each side of the trench about 6" beyond the proposed side of the trench so that where sheeting is required it will not rest against the remaining hard surface. The pavement can then be taken out with the backhoe as it moves along. In some cases it may be broken up and taken out by a front end loader so that the broken pavement will not be mixed with the other excavated material.

The trench is excavated to the width allowed by the specification to permit placing of the pipe and sheeting. The width of trench for payment when excavation is paid for by the cu. yd., is given in the specification. Any excavation beyond this limit is not paid for. If the contractor plans on excavating a wider trench to save on sheeting, he will have to take this into account in calculating the price. In any case the width of trench from the bottom of the bedding of pipe to the top of the pipe will have to be kept within the required limits. Generally on busy streets the width of trench is held to a minimum and shoring used. Where conditions permit it the contractor will widen the trench so that it will not be necessary to use sheeting.

The excavated material may be cast to one side of the trench and then replaced in the trench after the pipe is laid, by use of a front end loader. Rubber tired loaders are used where the surface is paved. If the working space is restricted the excavated material is loaded into trucks as it is excavated with some of the material being hauled away to a dumping area and sufficient is hauled around the working area to be deposited over the trench where the pipe has been partially backfilled.

DEWATERING SEWER TRENCH EXCAVATIONS

One of the largest problems encountered by contractors with the installation of sewers, is the dewatering of the sewer main excavation. Sewer pipe must be laid on a relatively dry bed. Minor dewatering consists of providing a sump hole alongside the line of sewer, a few feet below the invert, and pumping the water to the surface. When this type of dewatering is used the water is not excessive and the earth is usually fairly stable.

In conditions where the water table is high and the soil porous or a fine running sand, the removal of the water is a more serious problem. The most common and effective method of dewatering in this case is by wellpointing.

Wellpointing consists of jetting 1½"Ø pipes into the soil from the surface along the line of the sewer at from 2' - 5' intervals. The depth and spacing of the wellpoints depends on the water table, depth of excavation and type of soil. The depth and spacing must be such that at no time will the wellpoint screens be without water and loose their prime. Approximately 18' to 20' is the maximum practical distance below the pump suction that can be effectively drained using a single stage system. If deeper depths are required a multistage operation is required. The multistage operation consists of lines of wellpoints along the line of sewer at different elevations. A 6"Ø to 10"Ø header pipe is connected to the 1½Ø wellpoints and this in turn is connected to the pump. The pumping draws the water table down and the excavation can be done in dry soil.

SEWER PIPE BEDDING

The load which a sewer pipe will support is determined to a large extent on the way the sewer pipe is bedded. Thus the bedding of the pipe has considerable influence on the strength of the pipe which is required. The bedding of sewer pipe also varies with the type of soil encountered. In clay soils, without excessive water, it may only be required to carefully trim the trench bottom to the shape of the pipe. The pipe is then backfilled with selected material or granular material to a height of at least 1' - 0" above the top of the pipe. The material under and around the pipe should be shovel placed and shovel tamped to completely fill all voids. This type of installation is known as Class C bedding. Many contractors have found that the shaping of the bottom of the trench takes time and, therefore, is expensive. There is a growing practice to over excavate followed by backfilling with granular material. This brings us to our Class B type bedding. In Class B type bedding the sewer trench is over excavated. Fine granular material is installed as a base for the pipe. The pipe is then installed and the granular backfill is placed under and around the pipe to a height of about 1 foot above the pipe.

In very soft trench bottoms, it is desirable to over excavate and stabilize the soil by the addition of coarse gravel or rock. For those locations where a trench bottom cannot be satisfactorily stabilized with a rock bedding, a reinforced concrete cradle should be considered. The reinforced concrete cradle is classified as Class A bedding. The plain or reinforced concrete cradle extends up the side of the pipe to a distance equal to one quarter of the pipe diameter. If an unstable trench bottom extends for a considerable distance, it may be necessary to construct a concrete cradle supported by piling. The installation of a sewer main through a rock excavation is accomplished by bedding the sewer pipe in crushed stone. The size of the crushed stone varies with the size of the fizzures or crack in the rock. In all bedding requirements, it is necessary that the bedding is thoroughly compacted.

PLACEMENT OF PIPE

In most cases the sewer pipe is placed on the prepared bed by the backhoe which has dug the trench. The cable is attached to the bucket of the backhoe and the pipe lowered into the trench. The socket end of the pipe is laid upgrade. The larger diameter pipe is suspended by the backhoe just clear of the bedding and then the pipe is drawn or pushed into place for the proper completion of the joint. The smaller diameter pipe is forced into position by blocking the bell end with timber and using a steel bar as a lever to push the joint home. As the pipe size increases to approximately 12" to 24" it may be required to use the bucket of the backhoe as the force to complete

ıe joint. With larger diameter pipe say 30" and up, a come-long, or hand winch, should be used to ensure that the joint is ılled tight and also to hold it in position until the next pipe s placed. This method of pulling the pipe together, consists f anchoring the cable at the nearest manhole, running the cable ıside the pipe and through the winch and attaching it to the bell ıd of the pipe to be placed. By putting a tension on the cable ıe pipe is pushed into place. If the come-along is not used, the ırge diameter pipe have a tendency to spring back due to the ressure of the gasket against the pipe. The pipe should be kept ıug until the backfill load can safely hold the pipe in position.

After the pipe is placed on line and grade and the pipe ılled home, the bedding material is hand tamped under and along ıe pipe. The first foot of backfill material above the pipe is laced by hand in order not to damage the pipe. At the close of ıch days work the come-along should be left on and the end of ıe pipe protected with a close fitting stopper.

The blocking of the pipe to maintain the grade with imber or anything other than the specified bedding material is ot permitted.

If the pipe, gasket and installation is proper the oint should not require caulking to be water tight.

Poured in Place Concrete Sections

As the sewer diameter reaches 8' and up, it may become ore economical to use cast-in-place sections. Their construction s similar to that of a small building and is done in three lifts. irst, the floor or invert, second, the side walls and third the rch. The wooden forms and steel reinforcing is placed and the oncrete poured in the normal manner.

JNNELS

Where conformation of the ground or technical conditions ıll for it, and where there is the economic possibility, we resort o tunnelling. Lately, thanks to a big improvement in construction quipment, it has been found that tunnelling is more economical ıan the "Cut and cover" method in big cities where land is xpensive.

Tunnels are built for different purposes:- railroads, oads, water supply, sewers, hydro-electric power, etc.

In designing a tunnel, the first step is to determine ıe necessary cross-section of the tunnel. For OWRC purposes, s we are interested only in tunnels for water or sewage, it

would be necessary to determine what diameter of conduit should be placed in the tunnel for watersupply, or the shape of the cross-section for a tunnel to transport sewage.

The next step is to determine, by means of soil borings and diamond drilling in rock, the kind of ground which we will have in the underground work.

Having reliable data on subsoil conditions, and the cross-section established, we must then establish what would be the best method of tunnelling. Whether we should start working from one or from several headings depends on the length of the tunnel to be made and the length of time allowed for completion of the job.

In all but transportation tunnels (railroads, roads, navigable channels, etc.) we are forced to sink shafts, to have the perpendicular heading in front of us. Tunnels for water-supply and sewage must be constructed at a greater depth to correspond with the level of the pipelines. Shafts are usually rectangular in shape, as this has been proven a more economical method than constructing circular shafts. Shaft sinking is a science in itself and normally presents serious problems, the problems becoming more serious when the ground is soft and impregnated with water.

One of three methods may be used for tunnelling in soft ground. The boring method consists of drilling, by auger or other types of drilling apparatus, an opening into which a rigid steel pipe or reinforced concrete pipe can be pushed or pressed. This method is mostly used for tunnels of approximately 50 - 100 feet in length, to carry small diameter conduits (Up to three feet in diameter). It is employed where the ground is reasonably self-supporting, and where boulders or rock are rather scarce as these, when present, cause serious trouble.

The jacking method consists of propelling into the heading, by means of powerful jacks (Up to 500 tons), steel or reinforced concrete lining with a cutting edge on the first section. Excavation of the material from within the lining is done by hand, using tools driven by compressed air appropriate for the type of ground (Clay spades, picks, etc.). When the lining which becomes the tunnel is in place and the conduit in position inside, the space between the outside of the conduit and the liner must be backfilled. The backfilling is normally done by using sand backfill, grout or reinforced concrete. Due to very limited space the backfilling is done by means of placers driven by compressed air. This method is used for tunnels up to six feet in diameter as a rule, for distances up to 200 feet; however, there have been cases where lining of 10 ft diameter has been jacked for short stretches through good ground.

In the mining method, the kind of ground which the the tunnel will traverse governs the planning of its construction. If the ground is self-supporting even for a short period of time, it is possible to excavate (muck) and then place the necessary bracing or lining. The lining can be of wood, steel or reinforced concrete. If the ground is not self-supporting even for a very short period of time - in other words, "Not competent" in miners' terms - a shield may be used which is steadily pressed forward by means of jacks, under which shield the excavation and lining takes place. If the ground is not only incompetent but also waterbearing, compressed air is used to prevent the inflow of water.

Tunnels driven in rock may also vary. The rock may be competent, and when that is the case the operation goes forward smoothly, drilling, loading, blasting and mucking. After the tunnel is completed the lining operation follows. It is general practise to line with reinforced concrete. In incompetent rock the progress is much slower, as steel supports or steel lining must be used which are concreted in on completion of the excavation.

This, of course, is only a very brief introduction to tunnelling. The OWRC has been instrumental in the construction of two tunnels. The first is under the Grand River, to supply water to the industries in Port Maitland. This tunnel, 590 feet in length, shows a cross-section of 5' x 7'. The ground through which the tunnel was driven is sedimentary rock (limestone). Sulphite gas developed during the construction, but with the aid of more powerful ventilating equipment it was possible to finish this tunnel without the use of compressed air. Sixteen foot lengths of pressure pipe, 36" in diameter, was lowered through the westerly shaft and joined. The space between the pipes and the tunnel itself was filled with concrete containing sulphate resistant cement.

The second is a recently completed sewer tunnel which will serve the city of Sudbury as their intercepting sewer. Located approximately 100 feet below ground, the tunnel has an approximate length of 27,000 feet. There are six shafts which give access to the tunnel at different locations. The tunnel passed through sedimentary rock (Graywackie), and in some places through igneous Gabro. The rock is competent, and for this reason as well as the necessity for economy the tunnel was not lined; however, it was decided for hydraulic reasons to line the invert. The cross-section of this tunnel is 5' x 7', and will have a capacity of up to fifty million gallons per day.

MANHOLES

The principal purpose of manholes is to permit inspection, cleaning and the ventilation of sewers. The most common

manholes are built-in-place reinforced concrete and precast concrete sections. Both types are constructed on a base slab of approximately 8" thick. The minimum diameter is 3' with an access opening of at least 21". The benching of the bottom of the manhole must be such that the flow through is continuous and does not allow the deposition of solids. The manhole frame and cover is made adjustable in elevation by the use of three or four courses of brick between the frames and the concrete manhole. The spacing of the manholes is from 300' to 600' and depends on, the size of the sewer, changes of line and grade, and the intersection with other sewer lines. The criterion for the spacing of the manholes is the ability to operate sewer cleaning equipment. Arbitrary intervals have been adopted by municipalities and consultants such as, 150' maximum for small diameter sewers, 300' maximum for medium diameter sewers and 500' maximum where the pipe is large enough to walk through.

SERVICE CONNECTIONS

Tees or Y's should be provided for all house connections. The practice of breaking a hole into the side of a sewer and cementing a branch into it for a house connection should be avoided where possible. If it becomes necessary to break into a sewer, a proper clamp should be installed to insure an infiltration free connection. It is good practice, with deep sewer connections, to encase the service connection in concrete. The best type of encasement extends from a connection at the sewer main to a location where the service connection is laid on undisturbed ground.

BACKFILLING

Backfilling of a sewer trench is very important and should receive a good deal of attention.

The compaction of the backfill varies with the location of the sewer. On city streets, especially paved areas, the degree of compaction should be high. On less important streets or in sparsely inhabited subdivisions a more moderate specification for backfilling may be justified. Along outfall sewers in open country, it may be sufficient to mound the trench and after natural settlement regrade the area. By making the distinction in the degree of compaction in the specifications, the project costs are reduced.

The actual backfilling of the trench is accomplished in three lifts. The initial lift consists of hand placing and compacting the bedding or selected material, to an elevation 1' above the pipe. Satisfactory compaction is also accomplished by flooding or jetting of cohesionless soils such as sand or gravel, but care must be taken not to float the pipe. The

method of compacting clay and loam is to place backfill in thin layers (6") and compact with power equipment being careful not to disturb the pipe. The intermediate lift is the backfill from 1' above the pipe to within 18" of the surface. If the backfill material is cohesionless (sand or gravel) the backfilling is a one step machine operation followed by flooding or jetting to the required compaction. Clays used as backfill must be placed in layers (6") and compacted with power equipment. If the excavated material will not compact by these methods, other material must be imported. The final 18" should be dry material and slightly mounded over the trench area.

TESTING SEWERS AFTER INSTALLATION

The field tests performed on a sewer line consist of a sight or observation test, an alignment test, an infiltration or exfiltration test and observation by the use of a TV camera.

The sight or observation test is performed by sighting through the sewer, from manhole to manhole. This is done by shining a strong light through the sewer at one manhole and viewing the interior of the sewer from the adjacent manhole. In cases where the length between manholes or bends in the sewer line prevent direct sighting, mirrors can be used to both reflect the sun light or battery light into the pipeline and to view the inside of the pipe.

The alignment test is performed by pulling a wooden ball or similar object which is 2" less in diameter than the pipe, from manhole to manhole.

The exfiltration leakage test is used when there is no ground water pressure, or it is relatively low. The infiltration leakage test is used in areas where the ground water table is high.

Exfiltration leakage tests on sewers is usually performed after the pipe has been completely installed and back-filled. A section of the sewer is isolated between adjacent manholes by a watertight bulkhead. The test section of pipe is filled with water until there is a minimum head of 4' over the interior crown of the pipe at the highest point, providing the head on the lowest section does not exceed 15'. Precaution should be exercised to ensure that all air is removed. This would mean that the minimum pressure would be approximately 2# p.s.i. and the maximum pressure of approximately 6# p.s.i. After filling with water, the test section should be left for 24 hours to ensure that the absorption into the pipe wall is complete. After the 24 hour period the exfiltration test is performed with a minimum duration of one hour. This is accomplished by measuring the amount of water lost from the manhole at the highest location, during the one hour period.

At one of our projects the contractor elected to excavate the intake trench and dewater with pumps in the normal way. The trench was kept dry by this method and the pipe was installed and backfilled. It was later discovered that the pipe sank and broke in several places. The explanation given for this was that when the trench was excavated, the excavated earth removed the counter pressure for the artesian force. This tended to expand the soil at the invert of the pipe. When the pipe was laid and backfilled the counter pressure was again applied and the soil at the invert level was compressed. This settling did not happen equally and the pipe fractured. It was necessary to use the caisson method of dewatering to make the repairs.

SURFACE RESTORATION

After the backfill is completed it is necessary to restore the surface to a state as near as possible to its original condition. The specifications should set out what is to be done rather than just stating that the surface is to be returned to its original conditions. Complications arise where roads have been surface treated. If the trench is in the middle of the road it may be necessary to resurface the full width of road to obtain a good result. The arguments arise from the contractor claiming he is responsible only for the area over the trench, whereas his operations may have damaged the whole width. It is better in such a case to set out in the specification that the full width of road is to be resurfaced.

Where concrete or hot mix asphalt pavements are involved it is usuallly only necessary to repave the cut. Special attention should be given to proper consolidation of the backfill in the trench.

In the case of trenches across open fields, it is usual to mound the trench to compensate for the settlement of the backfill. The site should be cleaned up and left in a condition that the fields may be cultivated.

Infiltration testing is performed when the existing ground water level is a minimum of 4' above the interior crown of the pipe and does not exceed a maximum of 15'. The amount of water entering the isolated test section is usually pumped from the lowest manhole into a container and the quantity measured during a set period of time.

The allowable exfiltration or infiltration is from 300 to 500 gal./"Ø/mile of pipe/day, depending on the quality of sewer required. Expressed in gallons/"Ø per 100' of pipe/hour it is, from .236 to .395.

The examination of the interior of the pipe by a TV camera pulled through the pipe from manhole to manhole, is very effective but also very expensive. In this type of inspection the TV camera picks up the picture of the inside of the pipe and transfers it to a TV receiver at the surface.

SURFACE RESTORATION

The final restoration of the surface above the sewer trench has always been a problem. The problem stems from the fact that the original condition of the surface and the type of reinstatement must be determined before the contract goes out to tender. A statement such as "the surface should be restored to its original condition" is sometimes impossible to accomplish. Take the case where a road has been built up to a good surface by 12 or 15 years of annual oil or asphalt spraying. The road is actually a sprayed surface and yet through time has become a very substantial road. Unless the actual restoration has been specified for this particular road it is difficult to arrive at a satisfactory solution.

Another item which has given the Commission some difficulty is the restoration of an easement through private property. In many cases no actual record was made of the original condition of the property and existing shrubbery etc. The restoration then becomes one man's word against another's. The solution of this problem is to establish the replacement with the owner prior to the installation of the sewer.

INSTALLATION OF OUTFALL PIPE

A problem arises when installing pipe on the shore of a body of water, namely artesian water. This artesian effect is caused by the body of water causing an uplift pressure through the porous beach soil. One method of relieving this pressure is to excavate caissons some depth below the pipe invert and far enough away from the pipe centre line, not to interfere with the pipe installation. By pumping at these caissons the water pressure is relieved and the water removed. Pumping is continued until the pipe has been backfilled.

PREPARATION OF STOCK SOLUTIONS

FOR

SEWAGE TREATMENT PLANT LABORATORIES

by

W. W. Flynn

Laboratory Technician - OWRC

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Laboratory Technician - OWRC

INTRODUCTION

The efficient operation of a sewage treatment plant requires the performance of various chemical analyses, the accuracy of which is dependent on the equipment available and on the technical skill of the operator. It is the intent of this lecture to demonstrate to you one of the basic operations necessary for the performance of accurate analyses, and thus to encourage you to make full use of your own laboratory facilities, with the aim of increasing efficiency of treatment.

DEFINITIONS

An understanding of terms is essential before venturing into our subject, and thus the following definitions are given in the interest of clarity.

1. Solute: The solute is the substance that dissolves in the solvent.
2. Solvent: The solvent is the substance that does the dissolving.
3. Gram-atomic weight: The number of grams of an element equal to its atomic weight.
4. Molecular weight: The sum of the weight of all the individual atoms in the molecule, as represented by its formula.
5. Mole: The number of grams of a substance which is exactly equal to its molecular weight.
6. Molar solution (M): A solution containing 1 mole of solute per litre of solution.
7. Gram-equivalent weight: The weight in grams, of a substance that will furnish or react with 1 gram of hydrogen - or its equivalent.

8. Normal solution (N): A solution that contains one equivalent weight of solute per litre of solution.

TYPES OF SOLUTIONS

In the ordinary every-day work of a laboratory we are concerned with the amounts of various substances in samples. There are many methods of determ- these amounts, all based on chemical or physical properties (of atoms, molecules, or ions); but no matter which method we employ we continually have to prepare solutions of various substances for use in analyses.

The methods of expressing the concentrations of these solutions depend on the purpose for which the solution is required, but usually it is expressed as being either a dilute, concentrated, percentage, saturated, molar or a normal solution, and an explanation of the preparation of solutions of any of these different concentration units is the basis of this lecture.

DILUTE AND CONCENTRATED SOLUTIONS

The terms "dilute" and "concentrated" are widely used, and are indefinite as to the actual strength of a solution; they lack the precision which is necessary in describing solutions accurately. For example, a dilute solution of sodium chloride ($NaCl$) could be either 1 gm or 10 gms or even 30 gms of $NaCl$ per litre, while a concentrated solution of $NaCl$ may contain amounts in the region of 200 or 300 gms per litre.

It is possible, by adding more solute, or by evaporation, to make a dilute solution more concentrated. For example, to the $NaCl$ solution we can add more $NaCl$; or we can evaporate off some of the solvent - thus giving us the same quantity of $NaCl$ in a smaller volume of solution. Conversely a concentrated solution can be diluted by addition of solvent.

Thus it can be seen that these terms are quite vague as to concentration, and thus extreme accuracy in measurement of either solute or solvent is not necessary.

SATURATED SOLUTIONS

This is a term used to describe a solution in which there is dissolved as much of the solute as possible at a particular temperature. No matter how much more of the solute we add, we cannot put any more into solution - if the solution is already saturated. If the temperature is raised then the solubility of a given solute in a solvent is increased, and thus the solution will dissolve more solute. For example:-

Ordinary sugar is very soluble in water. If we add 204 grams of sugar to 100 mls of water at 20°C we have a saturated solution. If we increase the temperature we can add more sugar, which will come out of solution when the temperature drops, depositing crystals of sugar on the bottom of the container. The solution is saturated with the solute.

NOTE: Reference made to increase of solubility with increase in temperature does not hold for gases in solution, e.g., dissolved oxygen is less soluble at higher temperature.

PERCENTAGE SOLUTIONS

The expression of known percentage is the first of the methods of expressing concentration in definite units. It is usually based on the actual weight of solute in solution and is useful in determining the amount of solvent as well as solute. For example:

If we wish to make a 10 per cent solution of sodium hydroxide (NaOH) we need 10 gms of NaOH per 90 gms of solvent $\therefore$ for 250 gms of a 10 per cent NaOH solution we need $\frac{10}{100} \times 250 = 25$ gms of NaOH per 225 gms of H_2O.

It must be emphasized that percentages are sometimes expressed as grams/100 grams of solvent, mls/100 mls of solution, mls/100 mls solvent, and grams/100 mls of solution; but unless otherwise stated percentage means weight of solute per weight of solution. Perhaps the most common percentage expression used in laboratory work is the weight to volume percentage, which is normally expressed as a ratio, e.g., 18% W/V - meaning 18 gms of solute per 100 mls of solution or 180 gms of solute per 1000 mls of solution.

MOLAR SOLUTIONS (M)

The previous methods of expressing concentrations are called physical methods because they are based only on physical measurements of weight or volum and do not take into account the chemical properties of the solute. Thus none of these methods provides a direct measure of the chemical strength of a soluti This can be shown by the following example:

In the determination of alkalinity, hydrochloric acid (HCl) or sulphuric acid (H_2SO_4) can be used, but using 5 per cent solutions of both acids a greater volume of HCl than H_2SO_4 is necessary for the determination.

In order to do chemical calculations involving solutions, it is first necessary to convert the concentrations into chemical units. To eliminat this conversion, it is only necessary to state the concentration in chemical units when preparing the solutions.

The first of the chemical units employed for this purpose is called the mole, and a solution containing 1 mole of solute per litre of solution is defined as a molar solution. For example:

In preparing a 1.0 molar solution of NaOH it is necessary to weigh out 1.0 mole of NaOH, and from the definitions given:

1.0 mole of NaOH = number of grams of NaOH equal to the molecular weight of Na

$$= \underset{23}{\text{Na.}} + \underset{16}{\text{O.}} + \underset{1}{\text{H.}} \text{ (sum of atomic weights)}$$

= 40 gms.

$\therefore$ A 1.0 molar (1.0M) NaOH solution contains 40 gms NaOH per litre of solution and a 0.1 molar (0.1M) NaOH solution contains (40 x 0.1) gms = 4 gms NaOH per litre.

Similarly:

A 2.0M H_2SO_4 solution contains 2.0 moles of H_2SO_4 per litre of solution, and 2.0 moles H_2SO_4 = number of grams of H_2SO_4 equal to the molecular weight of H_2SO_4.

$$\begin{aligned} & \quad H_2 \qquad S. \qquad O_4 \\ &= (1 \times 2) + 32 + (16 \times 4) = 2 + 32 + 64 \\ &= 98 \text{ gms.} \end{aligned}$$

$\therefore$ A 2.0M H_2SO_4 solution contains (98 x 2.0) gms H_2SO_4 per litre of solution = 196 gms.

It should be noted that when preparing solutions of definite concentration, account should be taken of the percentage of the original solute. For example:

In preparing 1.0M nitric acid (HNO_3) we need 63 gms HNO_3 per litre of solution; but concentrated HNO_3 usually only contains 70 per cent HNO_3.

$\therefore$ To obtain 70 gms HNO_3 we need 100 gms conc. HNO_3 solution.

" " 1 " " " " $\frac{100}{70}$ " " " "

" " 63 " " " " $\frac{100}{70} \times 63$ " " "

$= \underline{\underline{90 \text{ gms}}}$

To obtain this amount we can either weigh out 90 gms or we can calculate the appropriate volume by making use of the specific gravity as follows:

The specific gravity of conc. HNO_3 = 1.5 which means that 1 ml weighs 1.5 gms

$\therefore$ To obtain 90 gms of conc. HNO_2 we measure $\frac{90}{1.5}$ mls. = 60 mls.

$\therefore$ A 1.0M solution of HNO_3 contains 90 <u>gms</u> of conc. HNO_3 per litre of solution or 60 <u>mls</u> conc. HNO_3 per litre of solution.

NORMAL SOLUTIONS (N)

The second method of expressing concentration in chemical units is called the "normal" method. It is the method most commonly used in analytical laboratory work because it is the most useful when calculating results of analyses; and because it is the simplest method of comparing the concentrations of various solutions. The "normal" method is based on the chemical equivalents of substances (the chemical activities of substances) - the idea being that solutions of the same "normality" will be of equal chemical strength.

For example, any solution of a concentration of 1.0 normality, (expressed as 1.0N) is chemically equivalent (equal in value) to any other solution of 1.0N.

However, the original calculations of the amounts of substances necessary to prepare "normal" solutions can be quite complicated. The definition given says that a "normal" solution contains one equivalent weight of solute per litre of solution; therefore, we must calculate the equivalent weight of a solute before we can prepare a "normal" solution. To determine the equivalent weight we must find how much of the substance will furnish or react with 1 gm of hydrogen, or its equivalent.

<u>For acids</u> the equivalent weight is calculated as the molecular weight divided by the number of active hydrogens (H) it will furnish per molecule, e.g:

(a) $\underline{H}Cl$ will furnish 1 hydrogen per molecule.

$\therefore$ the equivalent weight is $= \dfrac{\overset{H.}{1} + \overset{Cl.}{35.5}}{1} = \underline{\underline{36.5}}$ gms

(b) $\underline{H_2}SO_4$ will furnish 2 hydrogens per molecule.

$\therefore$ the equivalent weight is $\overset{H_2}{(1 \times 2)} + \overset{S}{32} + \overset{O_4}{(16 \times 4)} = \dfrac{98}{2} = \underline{\underline{49}}$ gms

<u>For bases</u> the equivalent weight is calculated as the molecular weight divided by the number of hydroxyls (OH) it will furnish per molecule (1 hydroxyl being chemically equivalent to 1 hydrogen), e.g:

(a) $Na\underline{OH}$ will furnish 1 hydroxyl per molecule.

$\therefore$ the equivalent weight is $\dfrac{\overset{Na}{23} + \overset{O.}{16} + \overset{H.}{1}}{1} = \underline{\underline{40}}$ gms

(b) $Ca(\underline{OH})_2$ will furnish 2 hydroxyls per molecule.

$\therefore$ the equivalent weight is $\overset{Ca}{40} + \overset{O_2}{(16 \times 2)} + \overset{H_2}{(1 \times 2)} = \dfrac{74}{2} = \underline{\underline{37}}$ gms

<u>For salts</u> (produced by reactions between acids and bases) the equivalent weight is calculated as the weight that will furnish <u>the equivalent</u> of 1 hydrogen or 1 hydroxyl, e.g:

(a) NaCl will furnish 1 sodium (Na) which is equivalent to 1 hydrogen.

$\therefore$ the equivalent weight of NaCl $= \dfrac{\overset{Na}{23} + \overset{Cl}{35.5}}{1} = \underline{\underline{58.5}}$ gms

(b) Na_2SO_4 (sodium sulphate) will furnish 2 sodiums which are equivalent to 2 hydrogens.

$\therefore$ the equivalent weight of $Na_2SO_4 = \dfrac{\overset{Na_2}{(23 \times 2)} + \overset{S}{32} + \overset{O_4}{(16 \times 4)}}{2} = \dfrac{142}{2} = \underline{\underline{71}}$ gms

(c) $AlCl_3$ (aluminum chloride) will furnish 3 chlorines (Cl) which are equivalei to 3 hydroxyls.

$\therefore$ the equivalent weight of $AlCl_3$ is $\dfrac{\overset{Al}{27} + \overset{Cl_3}{(35.5 \times 3)}}{3} = \dfrac{133.5}{3} = 44.5$ gms

It should be noted that the equivalent weight of a salt depends on the reaction in which it is involved, (at present it is not advisable to go into this particular calculation).

Having calculated the equivalent weight of the solute we must now determine how much we need to make a solution of known "normality". If the solution required is a 1.0N solution then we require 1.0 x equivalent weight of solute, in grams, per litre of solution. For example:

(a) To give us a 1.0N solution of H_2SO_4 we multiply the equivalent weight by 1.0 $\therefore$ 49 x 1.0 = 49

$\therefore$ A 1.0N solution of H_2SO_4 contains 49 gms H_2SO_4 per litre of solution.

(b) To give us a 0.2N solution of H_2SO_4 we multiply the equivalent weight by 0.2 = 49 x 0.2 = 9.8

$\therefore$ A 0.2N solution of H_2SO_4 contains 9.8 gms H_2SO_4 per litre of solution.

Thus we can readily see that in making a solution of 0.02N H_2SO_4, for alkalinity determinations, we need 49 x 0.02 gms H_2SO_4 = 0.98 gms H_2SO_4 per litre of solution. Again, as for molar solutions, we must take into account the strength of the acid (or any other substance) as given to us on the label of the container; and calculations should be made accordingly.

In the preparation of solutions of precise normality it must be remembered that the information given by the manufacturer is not always definite; the percentage of conc. H_2SO_4, for example, is given as being between 95% and 98%. Thus it is sometimes necessary, when preparing solutions for use in the laboratory, to prepare solutions of high normality, to approximate strength, then standardize, and dilute to the required normality. For very accurate work it is better to dilute the solutions to approximate normality then standardize, using recommended methods. This latter method is preferred because of the instability of some solutions through either evaporation, the effect of sunlight, oxidation, bacterial action, or general contamination, e.g., sodium thiosulphate is subject to deterioration by the action of bacteria and sunlight, therefore, a preservative is added to stock solutions to inhibit bacterial action, and the solution is kept in a dark-coloured bottle to inhibit decomposition by sunlight.

NOTES: The laboratory techniques employed in the preparation of stock solutions are mostly a matter of common sense combined with an elementary knowledge of the use of equipment. The following points should be kept in mind:

1. All equipment should be kept clean: all burettes, pipettes, and volumetric flasks should be washed with a cleaning solution as often as is necessary.

2. All stock solution containers should be properly labelled with the name of the solution, the concentration, the date of preparation and the name of the person preparing it.

3. Care should be taken that stock solutions are not contaminated by foreign substances from pipettes; thus, when it is necessary to obtain an amount of a stock solution it is better to pour from the container into a clean beaker; the aliquot then being taken from the beaker and the remaining solution in the beaker thrown away.

4. The use of a rubber bulb is recommended when using a pipette.

5. All corrosive solutions should be handled with extreme care; in the use of acid - always add the acid to water, never add water to acid.

6. In preparing solutions of definite concentration, make sure that chemicals are dried, according to the instructions given in the method, of analysis before weighing.

7. Solutions should be stored properly in the container best suited for them, e.g., sodium thiosulphate and silver nitrate should be stored in dark-coloured bottles.

CONCLUSION

The preparation of stock solutions is an elementary, but very necessary, part of all laboratory analyses. Some of you perhaps use solutions already prepared and sold as such; generally speaking it is less expensive to prepare your own. The real value in using the information given in this lecture is the increased confidence in performing analyses, plus the increased efficiency of treatment that results from analyses performed.

OPERATION OF AN UNDERLOADED ACTIVATED SLUDGE PLANT

by

C. H. Kretch

District Engineer - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 23, 1963

OPERATION OF AN UNDERLOADED

ACTIVATED SLUDGE PLANT

by

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District Engineer

INTRODUCTION

If an activated sludge plant is underloaded one might think that operation would be basically simple and problems would be minimal. However, this is not necessarily true. Because the activated sludge process is a biological process, the effect of prolonged retention and lower than normal loading factors on the secondary plant units particularly, is very much different than in a plant which is operating to its design capacity.

It is the purpose of this lecture to point out first of all in basic theoretical principles what is happening to the microorganisms during underloaded conditions and secondly how this affects operation of the plant in general.

TYPES OF UNDERLOADING CONDITIONS

The three main measurements of loading at any sewage treatment plant are flow, B.O.D. and suspended solids. Due to certain conditions within the sewer system such as high infiltration rates, combined sewers, number of pumping stations and presence of industrial wastes the three factors noted above may vary considerably with plant design figures. For example in the table below the flow, B.O.D., and suspended solids for six underloaded activated sludge plants in this province and one close to capacity are indicated as percentages of design.

It is noted that none of these seven plants have flows and B.O.D. loadings at corresponding percentages of design. It is therefore stressed that an operator should, through flow measurements and composite sampling have an accurate knowledge of the loading conditions at his plant particularly as related to the aeration section.

TABLE 1

Plant	Flow	B.O.D.	Suspended Solids
1 Lakeview	41%	27%	51%
11 Streetsville	51%	73%	64%
111 Huntsville	53%	40%	41%
1V Burlington E.G.	57%	98%	65%
V Georgetown	64%	43%	43%
V1 North Bay	78%	64%	85%
V11 Burlington D.L.	98%	80%	118%

BASIC THEORY OF THE ACTIVATED SLUDGE PROCESS

The stabilization of organic matter in any biological waste treatment system is brought about by the biochemical activities of the microorganisms in the system. In the activated sludge process bacteria are primarily responsible. Closely associated with the bacteria are protozoans among which one group, that of the ciliates, feeds upon bacteria. In the presence of abundant dissolved oxygen some forms of the bacteria and protozoa in the sewage die off or remain dormant while others multiply rapidly. By the action of their growth (synthesis) these organisms convert the pollutant absorbed on the sludge or dissolved in the sewage to carbon dioxide, water, sulphates, nitrates and the living protoplasm of their own bodies.

Growth Phases

The activated sludge process provides an active microbial culture and food for sustaining that culture. This system therefore follows the classical laws of cell growth and decay which consist of 5 phases as shown in Figure 1.

1. A lag phase producing little or no increase in bacterial numbers such as occurs when a slug of industrial wastes enters a plant and comes in contact with sludge acclimatized to domestic sewage only.

2. The log growth phase characterized by multiplication according to geometric progression. Here the food supply is unlimited and growth is limited only by the quantity of living microorganisms.

3. The declining growth phase in which growth becomes limited due to the lack of food.

4. A stationary or resting phase during which the peak population is maintained. This occurs when the available food supply diminishes and the rate of growth just equals the rate of cell death and destruction.

5. An endogenous respiration or death phase producing gradually diminishing bacterial numbers. The retardation of growth may be due either to the exhaustion of the food supply or the accumulation of waste products, which render the medium unfavourable for growth. Usually accumulation of waste products is more important than the exhaustion of food.

Oxidation - Synthesis Relationships

The mechanism of oxidation and synthesis is illustrated in Figure 11-A. McKinney has shown that the energy level of a biological system (lb.B.O.D. per day per lb. sludge) sometimes called the food: microorganism ratio (F:M) affects the proportion of B.O.D. oxidized, to that proportion transformed by synthesis to new protoplasm. This concept is illustrated graphically in Figure 11-B.

PRINCIPLES OF EXTENDED AERATION MODIFICATION

In recent years there has been a rapid increase in the number of small activated sludge plants for the treatment of domestic sewage and industrial wastes. Most of these plants have been constructed to operate on the principle of extended aeration in which the excess microbical masses formed are aerobically digested in the aeration tank. Since this modification may be adapted to an underloaded activated sludge plant there is some merit in discussing its advantages and limitations. Extended aeration plants which are sometimes referred to as "Aerobic Digestion" or "Total Oxidation" plants have the following common basis of design.

1. No primary treatment;

2. 24 hour aeration period in a completely mixed system;

3. Air supply 2 to 3 times greater than required for conventional activated sludge process treatment plants;

4. 4-hour settling period; and

5. Provision for sludge storage or removal

The extended aeration modification is designed to expose the incoming organic matter to as complete oxidation as possible by maintaining a high weight of active organisms in the system. The process allows the suspended solids to build-up in the system to a maximum value at which time the net or apparent growth of organisms in the system is balanced by the rate of unloading of organisms in the effluent. The sludge in this process spends a prolonged period in the endogenous respiration or death phase. Thus the sludge is older and less active, but a great number of organisms will be present. The endogenous respiration reduces the excess active mass in proportion to the sludge retention period which may be calculated as shown in Mr. Kay's paper on "Modifications of the Activated Sludge Process", Intermediate Sewage Works Course.

OPERATING DIFFICULTIES - EXTENDED AERATION PROCESS

In general, extensive operating problems have been encountered in the final settling tanks which are of two basic types. There is the so-called gravity flow sludge return type tank and the airlift or pump sludge return type. Both of these tanks have been designed primarily for low suspended solids, whereas the extended aeration plants have relatively high suspended solids.

When the loading reaches equilibrium problems are encountered due to denitrification. There is a very limited supply of dissolved oxygen present in the sedimentation tank which is used very quickly by the microorganisms. When this is gone, the organisms turn to the oxygen chemically combined with the nitrates formed in the aeration tank. When this oxygen is removed from the nitrates, some nitrogen gas is released and rises to the tank surface as a small bubble. When it rises, it lifts small particles of sludge to the tank surface which form a scum. The best control is rapid return of the activated sludge to the aeration tank.

Another problem in sedimentation tanks is grease and scum which is to be expected because raw sewage is fed to the system. Thus, in addition to the floating sludge and microorganisms, the tank becomes a grease and scum trap. If the surface of the tank is not kept clean, anaerobic conditions set in.

Excess sludge is another problem and there are three choices for handling it:

1. Discharge it with the plant effluent into the receiving stream. This is undesirable as solids may settle out in the stream and form sludge banks affecting the bottom fauna.

2. Dewater on a sand filter since sludge has been aerobically digested.

3. Haul away by tank truck for land disposal.

The need for sludge wasting is indicated by any of the following:

1. Aeration capacity of the system is exceeded.

2. Return sludge capacity is exceeded.

3. The final clarifier is upset by a high sludge return rate even though the rate is satisfactory.

APPLICATION OF EXTENDED AERATION MODIFICATION TO AN UNDERLOADED ACTIVATED SLUDGE PLANT

A conventional activated sludge plant is adaptable to the above modification if it is well underloaded, preferably has multiple units, a surplus of air supply, and ample return sludge capacity. If the final settling tanks have surface baffles or mechanical skimming equipment available it would be desirable to add the raw sewage directly to the aeration section and as a result, the primary tanks and sludge digester need not be placed into operation. The advantage would be offset by the increased air requirements. If a plant is underloaded on a 24 hour basis but receives extreme variations in hydraulic and organic loading the extended aeration modification would be better suited to handling these shock loadings. It appears to be basically a matter of economics as to the use of either method of operation.

To illustrate the adaption of an underloaded conventional activated sludge plant to the extended aeration modification, the following example is provided:

Example:

Assume a municipality owns an activated sludge plant with a design capacity of 1.0 M.G.D. and has the following basic treatment units.

2 - primary settling tanks (total retention at design flow - 1.5 hrs.)

2 - aeration sections (total retention - 6 hrs.)

2 - final settling tanks (total retention - 2 hrs.)

1 - sludge digester

1 - chlorine contact chamber

Return Sludge Pumping Capacity: normal - 250,000 G.P.D.
max. - 500,000 G.P.D.

Actual Loading - 250,000 G.P.D., that is, 1/4 of the design flow.

Operating conventionally: The operator would utilize one primary settling tank, one aeration section, one final settling tank, the chlorine contact chamber and the sludge digester.

If the raw sewage strength was 200 ppm B.O.D. and suspended solids and the primary treatment efficiency 35% and 60% removal of B.O.D. and suspended solids respectively, the mixed liquor suspended solids level would be 800 ppm resulting in a sludge age of 3.1 days. This represents an aeration tank loading of 0.325 lbs. of B.O.D. per lb. of MLSS.

Operating on the extended aeration modification: The operator would need not utilize either the primary settling tank nor the digester but would require both aeration sections and one final settling tank and the chlorine contact chamber. If 5000 ppm MLSS were maintained under these conditions, the sludge age would be 38 days and the F:M ratio would be .026 lbs of B.O.D. per lb. of MLSS. Return sludge pumping capacity would be available for 100 - 200% return.

The obvious advantages resulting from this modification would be the elimination of the primary settling and the sludge digestion facilities and the need for only periodic rather than regular sludge wasting procedures. The air requirements however would be increased. This arrangement would also handle shock loadings better whether hydraulic or organic.

SUMMARY AND CONCLUSIONS

It has been pointed our in this lecture that control of the activated sludge process when operated on the extended aeration modification differs considerably from that at a conventionally operated plant. Adaptation to this modification in an underloaded plant, if feasible, has some distinct advantages which result in simpler control. Reference should be made to Table 11 for a comparison of the two processes.

Assuming that a conventional activated sludge plant is severely underloaded and is adaptable to the extended aeration modification, the operator should consider the following points as most important to remember on this.

1. There is no need for primary settling nor sludge digestion facilities.

2. The aeration period and MLSS concentration are much greater resulting in lower B.O.D. to MLSS loading and prolonged sludge age.

3. Because of the much higher suspended solids level in the mixed liquor, the problem of excessive foaming in the aeration section should be much less than in a conventionally operated plant where the solids level is lower.

4. Sludge wasting is accomplished by wasting large volumes of activated sludge periodically rather than small amounts regularly.

5. Since the waste activated sludge has undergone aerobic digestion to a high degree, this material is relatively inoffensive and can be disposed of directly with little or no further treatment.

6. The extended aeration modification is better capable of handling hydraulic and/or organic shockloadings without deteriorating the quality of the plant effluent.

7. Operating difficulties associated with this modification of the activated sludge process are primarily as a result of the final settling tank and are as follows:

a. Solids rising from the tank bottom due to denitrification;

b. Excess solids escaping in the final effluent;

c. Grease and scum accumulations on the tank surface; and

d. Sludge wasting not governed by sludge age but by dissolved oxygen throughout secondary units, and final effluent quality.

Finally, the operator should be well aware of the loading conditions at his plant through accurate flow measurements and analysis results from composite samples.

Adequate laboratory facilities should be provided for the operator to control the various operating parameters as outlined in Table 11.

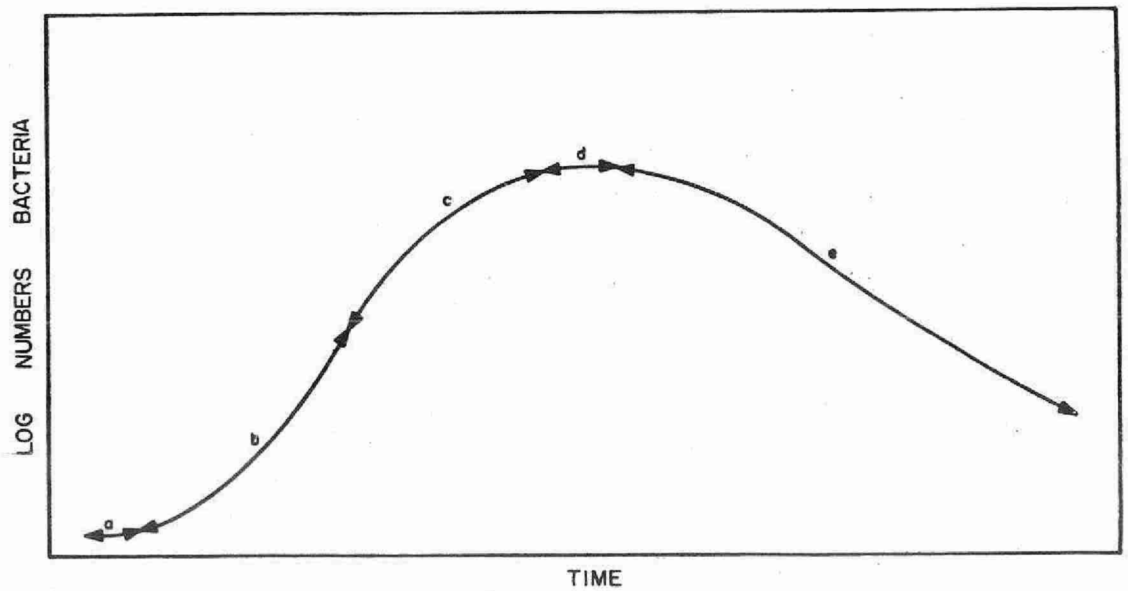

FIGURE 1 : BACTERIAL GROWTH CURVE

a ... LAG PHASE

b ... LOG GROWTH PHASE

c ... DECLINING GROWTH PHASE

d ... STATIONARY PHASE

e ... ENDOGENOUS RESPIRATION PHASE

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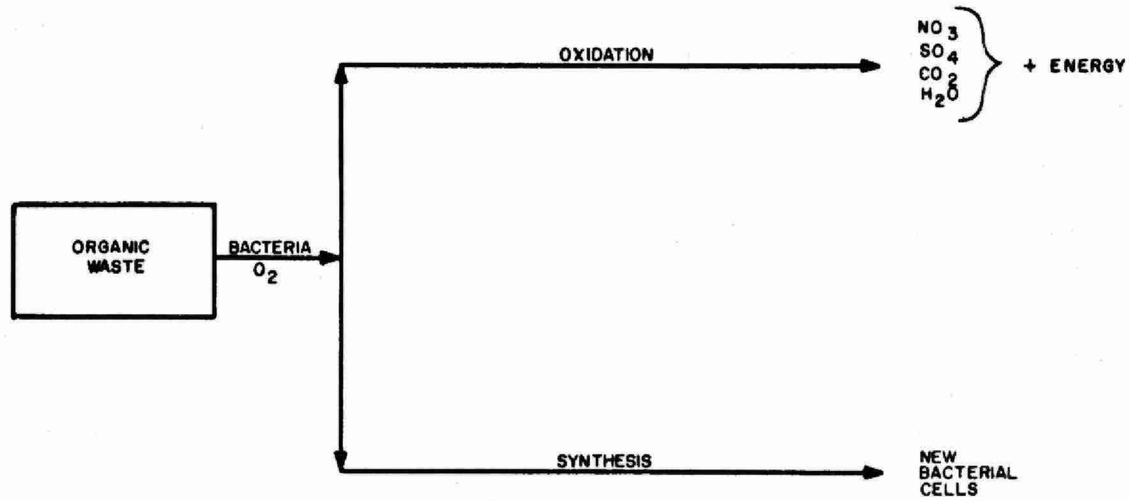

FIGURE II-A : MECHANISM OF OXIDATION AND SYNTHESIS

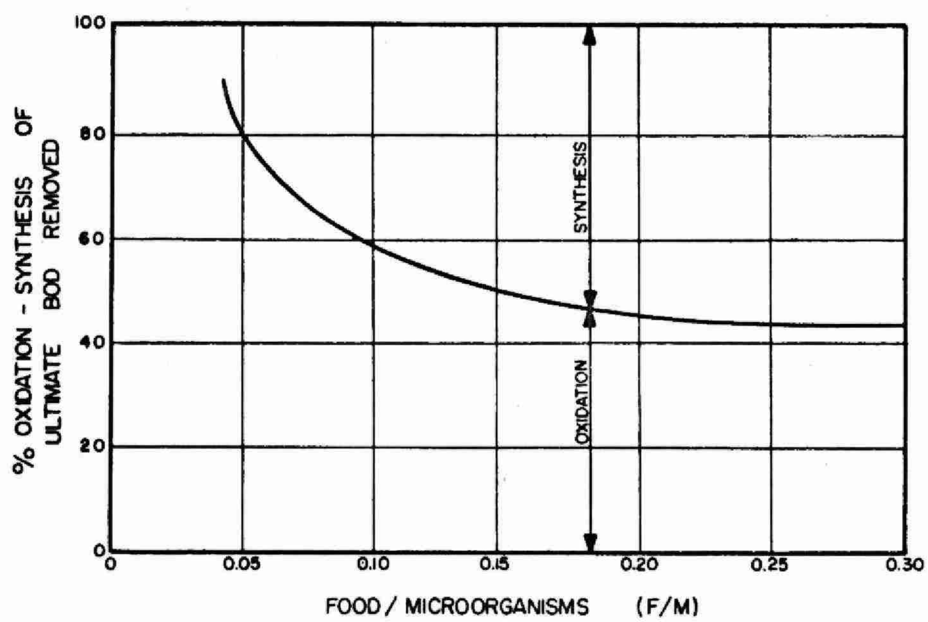

FIGURE II-B : RELATIONSHIP BETWEEN THE F/M RATIO & THE SYNTHESIS - OXIDATION RELATIONSHIP.

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TABLE 11

UNDERLOADED ACTIVATED SLUDGE PLANT

ITEM	CONVENTIONAL PROCESS	EXTENDED AERATION MODIFICATION
DESIGN:		
Primary Treatment	Yes	No
Anaerobic Sludge Digestion	Yes	No
Aeration Period	6-8 hrs.	24 hrs.
Final Settling Period	2 hrs.	4 hrs.
Air Requirements	approx. 1000 cu.ft./lb.B.O.D.	approx. 3000 cu.ft./lb. B.O.D.
Return Sludge Rate	25-50%	100-300%
OPERATION:		
lb. of B.O.D. to lbs. of MLSS	0.3 - 0.35	less than 0.05
Sludge Age	3.5 - 4 days	much higher than Conventional A.S. (not defined)
Total MLSS	1500-2500 ppm (diffused aeration)	5000-6000 ppm
	500 - 1200 ppm (mechanical aeration)	
% Vol. MLSS	65 - 85%	approx. 50%
S.V.I.	80 - 120	40 - 80
A.S. Wasting	Small amounts Regularly to Maintain Sludge Age of 3.5 to 4.0 days approx. 10% of Return Sludge	Larger volumes Periodically approx. 25 - 50% of MLSS

OPERATION OF AN OVERLOADED

ACTIVATED SLUDGE PLANT

by

G. H. Kay

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An Address To
The Ontario Water Resources Commission
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Toronto, Ontario
April 23, 1963

OPERATION OF AN OVERLOADED ACTIVATED SLUDGE PLANT

by

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Supervisor of Field Activities

Today we are to discuss the operation of an overloaded activated sludge plant. Basically, this consists of the same procedures as required to operate a properly loaded plant in the best possible manner, with further modifications to the plant processes, as required. Let us therefore, spend some time re-assessing the operations of a properly, or well-loaded plant.

Here under conditions of balanced design, absence of toxic sewage and nutrient unbalance etc., there are, in my opinion, three basic controls which if used with discretion, will allow proper effluents to accrue in most cases. These are, in the order of their considered importance:

(1) maintaining at all times, adequate dissolved oxygen in all sections of the aerators, final settling tanks and return sludge lines.

(2) removing the activated sludge from the final settling tanks at a rate at least approximately equivalent to its rate of deposition there.

(3) controlling the suspended solids concentration of the mixed liquor (M.L.S.S.) by maintaining a desired ratio of their total weight to the total daily weight of B.O.D. of the primary effluent. That is, with a known food-to-organism ratio, (F/M), or Sludge Age (M/F). This should be achieved by the rate of sludge wasting.

It should be realized that no absolute values can be stated for these that will apply equally for each plant. These depend on individual circumstances and therefore, the values to be given here for each of these are but guides from whence the operator may start to determine that modified value best suited for each plant.

These guides do allow the operator to have a technically sound approach to the operation and control of his plant. This will allow him to have confidence in his efforts so that the actual values pertinent to his plant can be resolved without prolonged delay.

DISSOLVED OXYGEN CONTENT (D.O)

In regard to (1) The Dissolved Oxygen Content, the process is biological in nature, so that if it is to be completed in a minimum of time and space or the maximum amount of sewage is to be treated, the action must not be inhibited or retarded by oxygen levels which are inadequate to sustain the optimum metabolism of the bacteria. It has been shown that this will not occur if minimum D.O. concentrations of approximately 0.5 - 1.0 ppm are available to all organisms, whether they be at the inlet or outlet ends of the aerator, in the final settling tank or in the return sludge line, since these are all part of the aerobic section of the plant. We have shown that in the conventional process, the demand is greatest at the inlet end of the aerator and can be provided best by initiating tapered aeration, or step aeration. To avoid depletion here and at any spots of poor circulation in the system, a minimum value of two ppm at the aerator outlet at all times is often recommended.

Values up to five ppm are common and higher values although yielding higher power costs, tend to guarantee avoidance of limiting conditions; i.e., less than .5 ppm of D.O. in any area. It is possible that some other organism would tend to predominate at higher values at the expense of the bacteria. Also, it is usual at these high values, that the effluent has most of its nitrites oxidized to nitrates. If the sludge is left in the final settling tanks for excessive periods, some micro-organisms will utilize these nitrates, releasing nitrogen gas which would tend to lift the sludge in clumps, over the weirs. This sludge loss could result also from another situation where nitrates are not available but due again to inadequate sludge return rates, the dissolved oxygen is depleted, septicity results, and the gas responsible for raising the sludge would then probably be methane. The "sludge-rising" condition accruing from these two situations would be different from "sludge-bulking" conditions.

Comment has been made that "over-aeration" meaning more particularly "excessive turbulence" may shear the floc excessively to yield reduced settling efficiencies and turbid effluents. It may be desirable however, to have this severe shearing to provide access for the oxygen to the centre of the floc. Generally then, the danger of excessive shearing of the floc has often been overemphasized. Therefore, when the rate of sludge return is adequate the danger to the process is definitely greater in excessively low D.O. values rather than in high D.O. values.

The term "over-oxidized" has been associated with high dissolved oxygen concentrations when pin-point floc is traversing the final settling tank weirs, and the suggestion is made that the sludge is "over-oxidized" as a result of the high dissolved oxygen values. Actually the pin-point floc usually consists of the cells of dead bacteria that result from the maintenance of excessive M.L.S.S. concentrations for the food supply available;

that is, the F/M ratio is too low. The sludge micro-organisms (M) have used up all the available food (F) or sewage, and have spent an excessive period in the endogenous respiration phase, utilizing all the food stored in their bodies. Here their metabolism rate is at a low level and so their need for oxygen is low. With equal spacing of diffusers, etc. in the tank, the dissolved oxygen concentration will therefore tend to be at a high level at its outlet end. High dissolved oxygen concentrations therefore, can be associated with, but are not the true cause of the "over-oxidized" or "burned-out" condition.

Where abundant air supply exists and the food available to the micro-organisms also exists in excess, extremely high dissolved oxygen values may be present and yet the sludge could be under-oxidized and bulking, due to an inadequate Sludge Age. This is due to the fact that the bacterial mass can use the available oxygen only up to a regulated maximum rate and this is predicated in part, by the average generation time, or time required for reproduction, of the micro-organisms of the sludge mass. An over-simplification of this statement suggests that other animals such as cattle, receiving adequate ventilation in a barn, do not become "over-oxidized" or "burned-out" if the oxygen concentration is increased by opening more windows in the barn.

Where highly nitrified effluents exist, it is very important to ensure adequate rates of sludge return to avoid denitrification in the final settling tank, with associated sludge rising in the form of clumps.

It is important to ensure an adequate minimum dissolved oxygen concentration in all parts of the system. Testing at various positions along the length of the aerator and at various points in the final settling tank will confirm or deny the adequacy of maintaining a minimum of two ppm at the outlet end. It is preferable to maintain an excess of dissolved oxygen to counteract the vacillations of flow conditions, for unless adequate minimum values of D.O. concentrations are present at all points at all times, the process is retarded and other tests at the plant are less meaningful.

Testing for dissolved oxygen should always be performed with the view of obtaining a truly representative result. In other words, the value of the laboratory result depends heavily on the integrity of the sample. This requires proper collection of the sample, usually from the aeration tank at the outlet end. Samples should not be collected from the aerator effluent channel where improper higher values will occur due to overflowing and turbulence at the outlet structure or weir of the tank. A sample at the outlet end collected approximately six hours after the period of maximum peak flow would probably be the most revealing of such samples.

Preferably, a proper D.O. sampler should be used; otherwise the sample should be collected slowly, with the bottle on its side to minimize the inclusion of atmospheric oxygen into the sample.

Therefore, until adequate dissolved oxygen is present in the aerator, final settling tank and return sludge lines, the process cannot reach its greatest efficiency nor treat as much sewage. Also, the value of the other tests are less meaningful.

RETURN SLUDGE

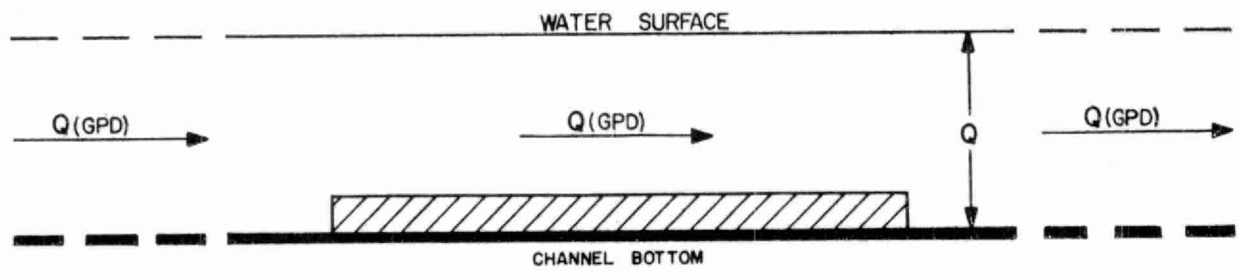
There are various methods of determining The Required Minimum Rate of Sludge Return (2) from the final settling tanks and some of these were set forth by Mr. Townshend in his Intermediate Course lecture.

In most plants it is often desirable to have a large number of reasonable results via a simple, easily-understood test rather than a single result from a test requiring complex computations that may result in extra errors and/or inadequate numbers of tests being performed. It will be my intent then, to develop one of the convenient methods of determining the required minimum rate of activated sludge return existing at a plant at any time of testing.

Whereas the term used is "Rate of Sludge Return" in order to assist you to follow my presentation, I would suggest that you consider the term to mean really, "Rate of Sludge Removal" (from the final settling tanks), for this is our primary concern. That is, due to the necessity of keeping the sludge aerobic to be in its best condition to later adsorb and oxidize the incoming sewage, it must continually be taken out of the final settling tank, which is not aerated, before it deteriorates due to insufficient dissolved oxygen. What is done with the sludge, that is, whether it is all returned to the aerator or not, is dependent on other considerations or tests and is not yet of concern in our presentation.

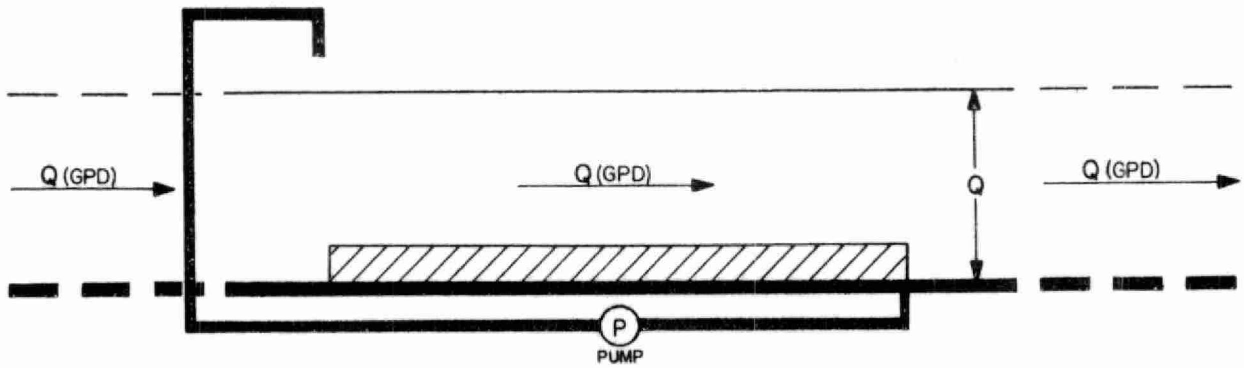
Therefore, while we discuss it here, I would suggest that you consider the rate required as a "Rate of Sludge Removal" (from the final settling tank).

To better assist us to comprehend required minimum rates of activated sludge return (or removal) let us digress for a moment and consider a channel with water flowing through it at a rate of Q, and thereby having a certain depth proportional to Q. This flow can be expressed in M.G.D., C.F.S., etc., and the depth can be expressed in inches, millimetres, etc. At this time, we are not concerned with the units to be used. Further, let us assume that in one section there is a slug of heavy material on the bottom of the channel being carried along in the flow, as per the diagram.



SIDE VIEW OR PROFILE

Now, if for some reason, it were to be desirable to retain the heavy material in the channel in approximately the same position, it could be achieved by having a hole or slot in the bottom of the channel at the downstream limit of the material, from whence a small tube and pump might discharge to the upstream limit of the material.



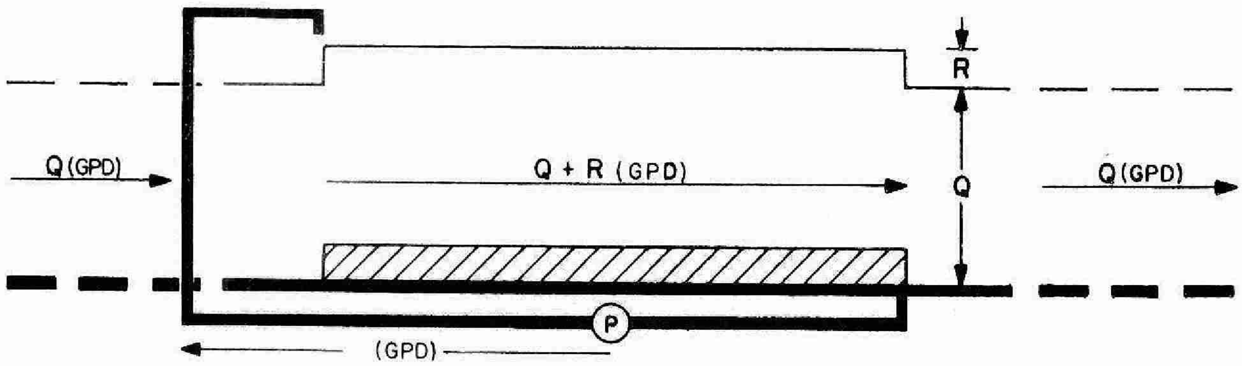
The problem at hand now, is to calculate the minimum rate of flow (quantity), induced in the pipe by the pump, that is required to return all of the heavy material. A rate higher than this required minimum will return a good deal of water with the material and flood the channel unnecessarily. A rate lower than this will be inadequate and some of the material will be lost downstream.

Regardless of the quantity of the recirculated or returned flow, it will merely be recirculating within the system between the original limits of the material, whereas Q still flows into one end of the channel, and therefore only Q flows out the other end. The flow in the section under consideration will be Q plus this rate of recirculation or return which we may call R (in G.P.D., C.F.S., etc.).

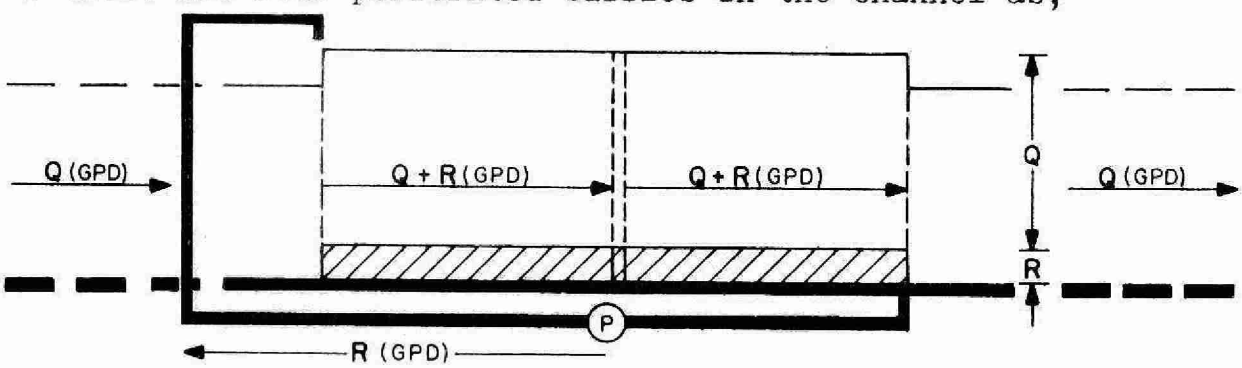
Some comment could be made here regarding the symbol Q being used for the <u>flow</u> and Q being used also for the <u>depth</u> of clear liquid. Similarly for the symbol R. Perhaps subscripts such as Qf and Qd as well as Rf and Rd or completely different capital letters might have been used. However, this might have produced further confusion. Q and R are just numbers like usual numerals

and the situation could occur where flows of Q in M.G.D. could produce depths of Q, exactly in feet, e.g., 1 M.G.D. = 1 ft. depth. The reader should remember that the value of the <u>depth</u> symbol by definition will be proportional to the value of the <u>flow</u> unit in each case and therefore he should not be excessively perturbed by any considered confliction.

Since the original depth was required for flows equal to Q, with the flow in the critical area increased by R, it is reasonable that the depth here will be increased proportional to R, or approximately by the depth of the layer of material.



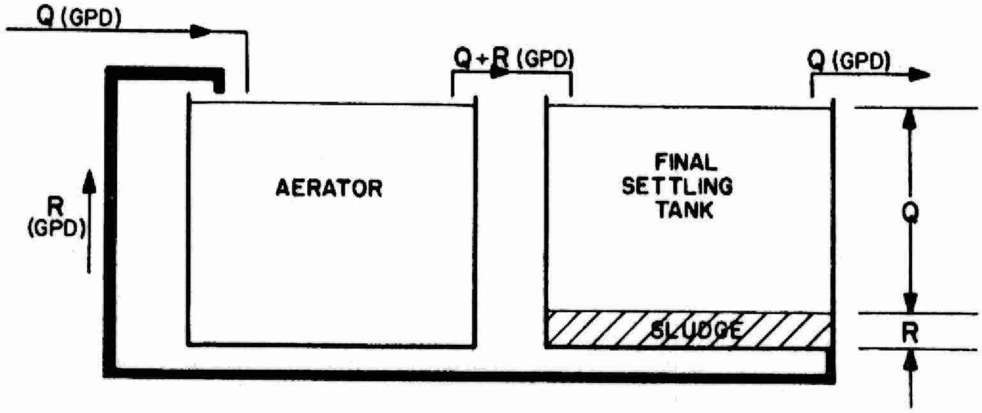
If there are four perforated baffles in the channel as,



then it resembles two tanks in series with a flow of Q passing into the first tank and out the last, with an added flow of R being recirculated or returned from the second to the first. The flow within the tanks is Q + R which is the situation that exists in an activated sludge plant, particularly at times when no return sludge is being wasted. This fact should be borne in mind in the later considerations of the Thirty-Minute Settling Test.

Therefore, before returning sludge, a total flow of Q enters the aerator and then flows to the final settling tank and is discharged. If for some reason we decide that we wish to retain the settled sludge in the system, we must return or recirculate at a rate equal to or greater than a required minimum rate of flow represented as R, or the sludge will be lost in the final effluent.

Then the flows in the system become



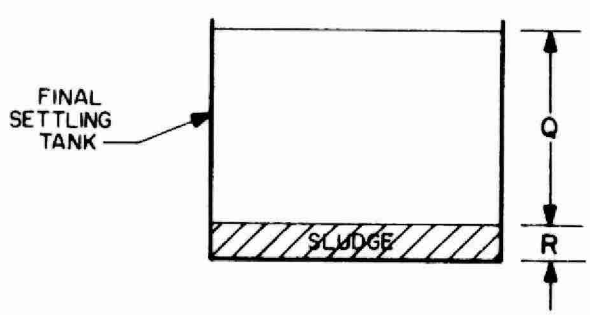
the sludge in the aerator is usually in suspension.

At the plant, the intent in setting a required minimum recirculation rate (R) in C.F.S., M.G.D., or other units, is to cause the so-called sludge blanket level to remain approximately at one position near the bottom of the final settling tank and this is recommended to be within approximately one foot from the bottom. An excessive R will cause the sludge to accumulate; the sludge blanket level will rise, and eventually cause sludge to flow over the outlet weirs. The bottom levels of sludge in the tank will deteriorate due to oxygen deficiency and when this material is returned to the aerator, it will not respond as activated sludge, but will result in a reduction in plant efficiency.

So it is desirable to have a rate of return that will avoid these problems.

The wide fluctuation in daily flows and strengths would require frequent rate-settings to hold the sludge at one level so that in practice, fewer changes are made with the realization that at times these will be inadequate and the depth of the sludge blanket will be increasing and at other times, more than adequate when the depth of sludge blanket will be decreasing, and eventually excess water may be returned with the sludge.

In the final settling tank, as in our example of the channel, the depth of the liquid therein represents the magnitude of the total flows. The depth of clear liquid represents the magnitude of the flow through the plant Q, and proportionally the depth of the sludge blanket represents the required minimum rate of sludge return R.



We would now desire to know what the rate of return R is in G.P.D., C.F.S., etc., or more particularly what fraction or percent it is of the plant flow Q, (G.P.D., C.F.S., etc.).

The Thirty-Minute Settling Test can be of assistance.

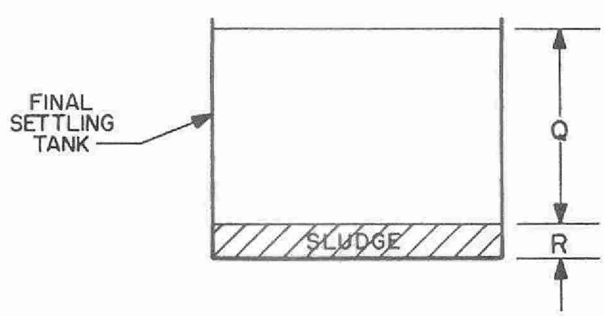
We use it, in part, to indicate relatively how the sludge is settling and compacting in the final settling tank. This is important since the activated sludge process is designed for this purpose. That is, to produce a rapid-settling floc from the colloidal, dissolved and finely divided materials of the primary effluent, that can then be separated from the sparkling effluent by sedimentation.

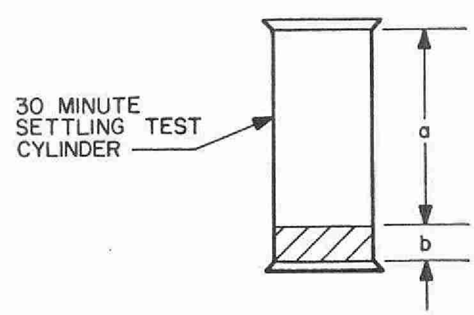
If the Thirty-Minute Settling Test mirrors the effect in the final tanks, why not use a time period of two hours similar to the usual design retention time of the final tanks, rather than the Thirty-Minute period? Well, the ideal settling conditions in the one-litre graduate provide approximately as much settling action as do most final tanks where inlet, outlet and density currents, etc., effect the sedimentation process. Also, the perishable nature of the sludge does not allow proper results when the detention period of the test is excessive. That is, the gases produced would tend to raise the settled sludge.

Times other than thirty minutes have been used and these may be very pertinent at individual plants; but to allow comparison of the results of this test and sludge indices, the time was standardized at thirty minutes, by Mohlman.

Therefore, in the one-litre graduate, we can acquire equivalent settling effects and thereby sludge depths approximately proportional to those in the final settling tanks.

Let us represent the depth of sludge settling in the test (ml.) by "b" and the depth of clear liquid above it by "a".





Then if all our assumptions are acceptable we can write the ratios

$$\frac{R}{R + Q} = \frac{b}{a + b}$$

Also, we can say that R will be some fraction of Q, such as ½ or ¼ etc., so let us call this fraction that R is of Q = r, and then R = rQ or Qr. By substituting, the above ratio becomes

$$\frac{Qr}{Qr + Q} = \frac{b}{a + b}$$

$$\frac{(Q)r}{(Q)[r + 1]} = \frac{b}{a + b}$$

$$\frac{r}{r + 1} = \frac{b}{a + b}$$

by cross multiplying

$$ra + rb = rb + b$$

by subtraction

$$ra = b$$

$$r = \frac{b}{a}$$

Since we can obtain values for "a" and "b" from the Thirty-Minute Settling Test, we can obtain the required minimum rate of sludge return, stated as a fraction of Q, at each time the Thirty-Minute Settling Test is performed.

Example

Number of ml. of sludge settling in a 1,000 c.c. cylinder in thirty minutes = 210

$\therefore$ b = 210 a = 1,000 - b = 1,000 - 210 = 790

$$r = \frac{b}{a} = \frac{210}{790} = .27$$

$\therefore$ by our definition "r" is the fraction the rate of return (R)

is of the flow (Q) $= \frac{R}{Q} = .27$ or $\frac{27}{100}$ of Q

If this is multiplied by 100, we obtain a percentage and the required rate of return R is 27% of Q.

This value will always be greater than the result of the Thirty-Minute Settling Test expressed as a percent, which in this case was 21%, because it really represents 21% of the total of sewage flow, plus return sludge flow (Q + R). This is equal to 27% of Q alone, and Q is a value we have or can obtain.

The plant operator who utilizes the results of the Thirty-Minute Settling Test (expressed as a percent) unaltered for his value of "r" rather than values obtained by a formula such as that under discussion here, would have an increasing depth of sludge blanket with deleterious effects such as "sludge-bulking", "sludge-rising" and retarded aerobic processes. All of which are deleterious to any plant, but particularly to the over-loaded plant.

The sample for this test should always be collected at the outlet end of the aerator in the conventional and modified processes. The sludge return devices must be capable of returning at this rate. If the sludge volume index frequently rises excessively so that return rates beyond this capacity are required, it may be necessary to acquire further sludge return units. It may be possible to improve the situation temporarily by changing the pump impellor or rewinding the pump motors, unless process changes are made. If inadequate rates continue to exist reduced total effluent quality must accrue due to necessary by-passing of sewage or reduction of Sludge Age.

It can be seen that this formula is basically the first formula of exhibit 1, page 75, of Mr. Townshend's previous lecture.

As previously stated, conditions will vary from plant to plant, and the propriety of this formula at each of these will vary slightly. It is recommended therefore, that it be assessed by frequently determining the sludge blanket depth in the final tank. This can be done by lowering a stoppered bottle attached to a pole to the lower levels of the tank. After the cork is removed with a string, the quality of the sample can be assessed. The sludge blanket should not be more than approximately one foot in depth.

It should be remembered that this rate is the required minimum rate of sludge return at the time of testing. More preferably, it is the required minimum rate of sludge removal since no mention has been made or needed to be made for this discussion in regard to the amount of sludge to be returned to the aerator, nor the amount of sludge to be wasted. This quantity will be discussed in the next section (3).

MIXED LIQUOR SUSPENDED SOLIDS (3)

Activated sludge research teams have confirmed what bacteriologists have known for some time, that in biological systems such as at activated sludge plants, the organisms therein, as with most animals, thrive best when supplied food at a certain rate. The food (F) in this case is, primarily, the bio-degradable organic material of the sewage or primary effluent.

Since oxygen is required by the bacteria to biologically degrade this food, and since some organic material is non-degradable, the B.O.D. value is used to indicate better the load of biodegradable organic material.

The process consists of having the animals or micro-organisms (M) use this food in their natural biological processes and so oxidize it ultimately to basically stable gases, water and minerals. When entering the final settling tank, the animals in the flocs should be in a form conducive to rapid settling for separation from the final effluent liquids.

It has been found that the micro-organisms (M) as determined by convenient methods, respond best in many cases, when receiving approximately one-quarter of their weight in food per day. In the case of sewage treatment, the average weight of food (F) in the form of sewage is predetermined by sewage flows and strengths, and therefore the problem is really how many, or what weight of animals or micro-organisms (M) should be retained in the aerator to provide optimum operation. That is, we wish to keep the F/M ratio or food (sewage) - to-micro-organisms (bacteria) ratio, as determined by convenient methods, equal to perhaps approximately ¼ or .25 by controlling the numbers or weight of the suspended solids or bacteria in the aerator.

The total weight of B.O.D. of the daily sewage flow (F) is compared to the total weight of the suspended solids in the aerator (M) to obtain this ratio. To obtain these values requires the determination of the concentration or the percent or ppm of the B.O.D. of the primary effluent and also of the suspended solids of the mixed liquor. When multiplied by the weight of the liquid quantities, the weights of food and micro-organisms can be calculated and the ratio between them obtained.

Example

If the B.O.D. concentrations of the primary effluents have been determined by analyses of samples sent to the Commission Laboratory and these average 140 ppm or

$$\frac{140}{1,000,000} = .014\% \text{ of the weight of the primary effluent,}$$

a flow of 1 M.G.D., will produce

$$\frac{140}{1,000,000} \times 1,000,000 \times 10 = 1400 \text{ lbs.}$$

of B.O.D. or food (F) being fed each day to the aerator. If the concentration of suspended solids in the aerator is calculated by centrifuge and graph or by drying, to be 2,500 ppm or

$$\frac{2,500}{1,000,000} = .25\% \text{ of the weight of the aerator contents,}$$

and the capacity of the aerator is 300,000 imperial gallons, there will be $\frac{2500}{1,000,000} \times 300,000 \times 10 = 7500$ lbs. of suspended solids and thereby we may suggest 7500 lbs. of micro-organisms (M) in the aerator. Therefore, the food to micro-organism ratio (F/M) would be $\frac{F}{M} = \frac{1400}{7500} = .19$ or approximately 1/5.

If as we have said, we wish to operate the plant at an F/M ratio = .25 or ¼, that is, if we wish to provide the animals with ¼ of their weight in food each day, since F is already determined by the sewage flow and strength, we must decrease M. Therefore, the weight of solids in the aerator should be reduced slowly by wasting a greater portion of the return sludge until **their** concentration approximates $2500 \times \frac{.19}{.25} = 1900$ ppm.

The total weight of M.L.S.S. will then be $\frac{1900}{1,000,000} \times 300,000 \times 10$

$= 5700$ lbs.

The $\frac{F}{M}$ ratio is then $\frac{1400}{5700} = .25$

To the operator therefore the prime use of the F.M ratio is to control the concentration of M.L.S.S. at a level to produce an **F/M** ratio optimum for that plant, by the rate of wasting of return sludge.

SLUDGE AGE

The term Sludge Age has been used to describe another operating control. Basically, it is equal to the reciprocal of the **F/M** ratio or $\frac{1}{F/M} = M/F$, and its results are reported as "days".

This suggests that at certain rates of sewage feeding and M.L.S.S. concentrations, since the food is converted into micro-organisms, each bacteria is apparently retained in the system of the aerator and final settling tank on the average, for a number of days before being discharged in the effluent or in the waste sludge. That is, for our recommended food-to-micro-organism ratio value or F/M ratio = ¼ or .25, the Sludge Age would be $\frac{1}{F/M} = \frac{1}{.25}$ or 4 days.

FOOD (F)

The Sludge Age originally, was based on the <u>suspended solids</u> of the primary effluent. Sludge Age (days)

$$= \frac{\text{p.p.m. of aerator suspended solids} \times \text{aerator capacity (gals)} \times 10}{\text{p.p.m. of suspended solids of primary effluent} \times Q\ \text{(G.P.D.)} \times 10}$$

Where the concentration (p.p.m.) of suspended solids and B.O.D. (p.p.m.) in the primary effluent are similar, as in most domestic sewages, there is relatively no error. However, in today's sewages with a high proportion of industrial wastes, containing a high percentage of the B.O.D. in the dissolved form, these may not be similar. Since the food (F) of the bacteria is primarily the degradable organic material (and which is indicated better by the B.O.D. values), to have the Sludge Age meaningful, it becomes more necessary to substitute the B.O.D. value of the primary effluent for the suspended solids value thereof. Then the Sludge Age becomes exactly the reciprocal of the F/M ratio or $\frac{1}{F/M} = \frac{M}{F}$

$$= \frac{\text{p.p.m. of aerator suspended solids x aerator capacity (gals) x 10}}{\text{p.p.m. of B.O.D. of primary effluent x Q (G.P.D.) x 10}}$$

MICRO-ORGANISMS (M)

The weight of the aerator suspended solids are used to represent the weight of the micro-organisms (M). Actually, we are really interested in the weight of the <u>active</u> organisms that can adsorb or clarify putrescible organic solids of the primary effluent, oxidize same, and be settled later in the final tanks.

The aerator suspended solids (M.L.S.S.) consist of active micro-organisms, dead micro-organisms, insoluble inorganic material, insoluble non-degradable organics, etc. It might be more accurate to utilize the weight of volatile suspended solids of the M.L.S.S.; however, even in normal activated sludge, it is estimated that only 25-50 percent of the volatile mixed liquor suspended solids are made up of living organisms. It would be difficult to ascertain the quantity of active organisms in the M.L.S.S. However, despite these recognized limitations, the Sludge Age (M/F) or the F/M ratio are both of extreme value in operating a plant at its maximum efficiency, and do in particular provide technically sound means of controlling the rate of wasting of the return sludge. Regular continuous wasting should be attempted to avoid imparting shock conditions.

The volatile content of the M.L.S.S. should approximate 60-75 percent thereof. When this value approaches the lower limit of this range, if apparently a proper Sludge Age is being maintained, it suggests that excess inorganic material is present. A higher-than-usual apparent Sludge Age should be used or a temporary rapid wasting of 25-50 percent of the M.L.S.S. should be effected to remove these inorganics.

In a high-Sludge-Age-system resembling the Extended Aeration Modification, a high percentage of volatile solids may be present, but with proper effluents accruing. This can be attributed to the presence of few inorganic solids and a high percentage of non-degradable volatile organics as will build up an "old" extended aeration system. Temporary rapid wasting is recommended here also.

It should be reiterated that the B.O.D. of the primary effluent is more representative than the Suspended Solids in the calculation of the Sludge Age.

Sludges Ages of $3\frac{1}{2}$ - 4 days have provided excellent effluent qualities at a good number of plants. Starting from these values the operator should vary the Sludge Age at his own plant to ascertain that age which consistently provides the best effluent qualities.

Since this value is dependent on the activity of the micro-organisms (M) and since temperature has a marked effect thereon, it is reasonable to expect that due to reduced activity during periods of cold weather, greater numbers of organisms and thereby higher Sludge Age values should be carried. Where a Sludge Age of four days has proven proper in the summer months, a Sludge Age approximating five days during the winter months might be expected.

MECHANICAL AERATION

Some literature suggests M.L.S.S. concentration ranges of 1200 - 3000 ppm in diffused aeration plants and 500 - 1200 ppm in mechanical aeration plants. This lower range is not caused by any shortcoming in the efficiency of mechanical aeration. It is actually because the relative possibility of short-circuiting in a single mechanical aeration tank is much greater than in a long diffused aeration tank, and thereby poor effluents could accrue if the same level of M.L.S.S. concentrations were carried in the single mechanical aeration tanks. Where several mechanical aerators are operated in series, the above-suggested range of 500-1200 ppm should not be used since the probability of short-circuiting is reduced and the range of 1200 - 3000 is now proper.

In either mechanical aeration or diffused air plants, the Sludge Age primarily should be used to control M.L.S.S. concentrations rather than any suggested solids concentration range. It can be seen therefore, that the single mechanical aerator installations may require longer retention periods (8-12 hours), or larger tanks, to satisfy the Sludge Age concept and keep the M.L.S.S. concentrations within this suggested range to avoid excessive short-circuiting.

FOAMING

At some diffused air plants where foaming is a problem in the aerator, the M.L.S.S. concentrations are sometimes maintained at a level higher than that required by the process, in order to minimize the foam. This results in an appreciably longer Sludge Age. This will result in higher aeration costs since some of the sludge is being digested aerobically, usually with associated higher suspended solids values in the plant effluent. It is much more preferable to control the M.L.S.S. to suit the best Sludge Age at the plant and control the foam by water sprays or non-objectionable chemicals. The resulting reduction of Sludge Age could assist an apparently overloaded plant.

DIGESTER SUPERNATANT

In calculating Sludge Ages and F/M ratios, it should be borne in mind that the supernatant from the digester will be adding an appreciable B.O.D. or (F) to the aerator and could produce a slightly lower true Sludge Age than the apparent Sludge Age based only on the B.O.D. of the primary effluent. Realistically, this is not usually used in these calculations. However, the important aspect is to know of this added loading, and to operate in a manner to return this flow at a low rate in order not to upset the plant appreciably. Where digesters are being operated full, this requires the pumping of raw sludge to be performed slowly at frequent intervals, preferably at off-peak periods.

The quality of the supernatant is therefore important to the aeration section. Selection of optimum supernatant levels is required and where digester mixing mechanisms are in use, these should be stopped for a period preferably of several hours prior to waste sludge pumping and subsequent supernatant discharge, to allow a quiescent period and so allow an improved supernatant quality to prevail.

Some operators have attempted to control the rate of sludge wasting and thereby the M.L.S.S. concentration, by the use of the Thirty-Minute Settling Test alone; that is, by controlling the quantity settling in this test at perhaps 200 ml., by sludge wasting. However, this is but a volumetric value, and will be pertinent only if the quality of the sludge is relatively unchanged. Actually, the quality of the sludge is usually changing continually. That is, if the M.L.S.S. concentrations were kept constant, values for the Thirty-Minute Test could range from a minimum to 100%. On seeing this value rise, the unskilled operator could feel that more sludge should be wasted. This is commonly the opposite to what is required since it would lower the Sludge Age and could ultimately cause the process to fail.

Where an average S.V.I. of 100 exists, the daily values may vary from 40 - 400; if however, the S.V.I. at a plant somehow remained reasonably constant, the Thirty Minute Settling Test could by itself still be of assistance in M.L.S.S. control.

Example

If by some strange faculty, the S.V.I. at a plant remained approximately at 100 and the optimum M.L.S.S. concentration from the Sludge Age was 2,000 ppm, sludge wasting could be controlled to retain the Thirty-Minute Settling Test values at 20%. This steady S.V.I. is not reasonable and therefore the M.L.S.S. concentrations should be controlled by the Sludge Age or by the F/M ratio rather than the Thirty-Minute Settling Test alone.

RELIEVING ADVERSE LOADING ON THE PLANT

The domestic wastes of our sewered municipalities are water-borne via plumbing and sewerage, to the disposal plant. This requires all the water that is used to transport these wastes, and that is now polluted, to be treated at the plant. To forestall the day that plant enlargements are required, and to guard the existing capacity of the plant, every effort should be made to prevent extraneous water from arriving in the sewers at the plant.

Domestic water meters will prevent irresponsible excess water usage. Roof downspouts should not be allowed to be connected to a sanitary sewer system. Separate sanitary and storm sewers are desirable. Sewers should be constructed in a manner to limit infiltration. Checks should be made for such infiltration in existing sewers near watercourses. Where combined sewer overflow structures exist, these should be checked periodically. Sewer use by-laws should encourage control of waste flows at commercial and industrial sources. Where sluggish sewers exist, these should be flushed periodically to avoid having the settled septic material arrive at the plant in a slug after a rainfall. A continuous sewer cleaning program should be in effect partially for the same reason.

Pumping stations should be built and controlled to avoid septicity and prolonged surges in the flows. Screens should be cleaned at frequent intervals. Primary sludge should be removed slowly at frequent intervals during each day. A sampling tap or other device to assess the thickness of this sludge is desirable. If multiple units exist, equal flow should be ensured. If the digester supernatant creates excessive septic conditions in the primary tanks, consideration should be given to discharging same to the aerator. Short-circuiting in the primary tanks should be assessed and corrected with baffles, if necessary. Scum should be removed before it becomes excessive.

It is of prime importance for each operator to know the fluctuations in hourly flows at his plant. In most plants there is at least one excessive peak flow period and this usually occurs during the daylight hours. The flows during the night hours at most plants are usually less than the design flows. It sometimes comes as a real revelation to the operator when he is first made aware of the low magnitude of these night flows. It is because of this night period which allows rest or regeneration, that the sludge which may have been bulking at three o'clock on the previous afternoon, is in a good condition on the following morning.

Mr. McTavish will be illustrating some graphs showing these severe fluctuations.

Similarly, the strength of the sewage usually decreases during the night. It is not uncommon to have dry weather night flows with B.O.D. values of only 80 ppm. From our food to micro-organism (F/M) ratio discussions, it is obvious that the food (F)

value will be reduced dramatically during the night hours at most plants. If the sludge is in a good condition in the late afternoon, there may be inadequate food during the night and it will be in the endogenous phase for an excessive period so that it will be partially "burned out" and will not be in an optimum active state to process the sewage flows next morning. Its volume will be at a low level. Sludges bulking in the afternoon will have stored food for the night and under similar conditions should be in a good condition next morning.

The changing S.V.I.'s and thereby sludge volumes will necessitate changes of sludge return rates. The volume of sludge during the night will ordinarily be reduced so that if the daytime rate of return is not changed, excessive quantities of water will be returned, using up the capacity of the aerator. As a minimum, this rate should be checked and adjusted if necessary, at the start and end of each operating shift.

Where more than one final settling tank is in use, tests should be made on the individual tanks and rates of flow and sludge return adjusted to suit. These tanks should also be checked for short-circuiting and excessive currents. Weir levels should be uniform.

PROCESS MODIFICATIONS

If all good operating techniques have been inaugurated, including primarily the three basic controls mentioned in this lecture with no limiting aeration or return sludge facilities nor nutrient unbalance or toxic sewage conditions existing, where poor effluents due to overloading persist, process changes will be re-required. Naturally, the easiest solution would probably be the addition of more facilities. However, if only the existing units are to be used, conversion to one of the modifications of the process is indicated.

Each of these will be governed in part by the sewage flow and strength fluctuations at the plant under consideration.

I have again included Figure 1 "Variation of Five-Day B.O.D. of Raw Sewage-Activated Sludge Mixtures with Aeration Period". This applies to primary effluents as well as raw sewage. The phenomena of the B.O.D. being reduced to provide acceptable effluents after a minimum of thirty minutes contact is shown. This does not pertain to the same extent to sewages having a high percentage of the B.O.D. in the dissolved form for it is primarily the colloidal and finely divided sewage solids which are adsorbed by some bacteria of the sludge in this time. It would seem that the extra waste products resulting from the metabolism of these bacteria will not yet have been released to raise the B.O.D. level again. These new waste products may be altered in turn by the various other strains of bacteria and micro-organisms until the level of B.O.D. in the liquid once again, preferably at the outlet

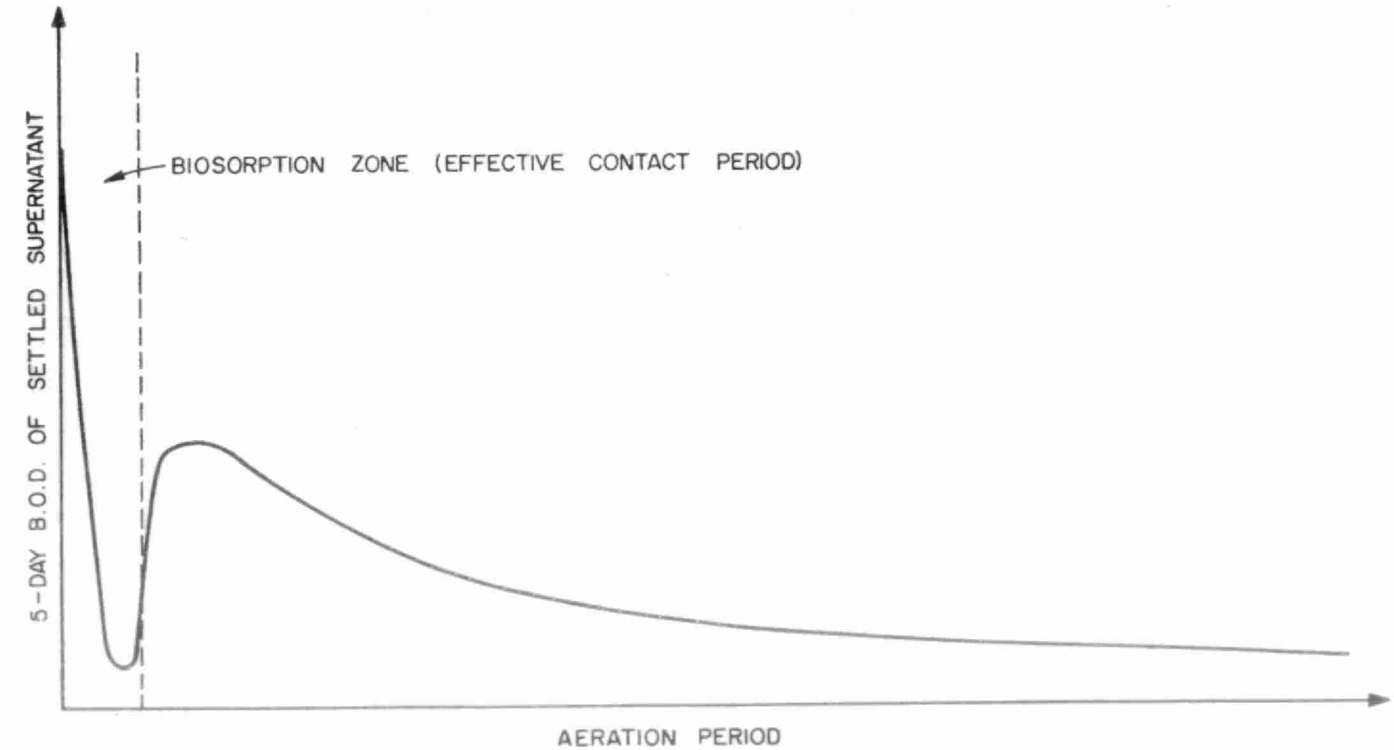


FIG. I VARIATION OF 5-DAY B.O.D. OF RAW SEWAGE-ACTIVATED SLUDGE MIXTURE WITH AERATION PERIOD.

end of the aerator, will have been lowered to a level which allows discharge of proper effluents. It appears then, that each strain of bacteria performs a specific job so that after an initial adsorption period, the reduction of the B.O.D. of the liquid is a step-by-step process.

Together with associated adequate conditioning or oxidation of the sludge, this is the basis of the Sludge Reareation modification. The Step Aeration modification can also utilize this theory. We have previously shown that both of these modifications can result in a reduction of required aeration capacity, and thereby allow overloaded plants to process above-original-design flows, to provide proper effluents.

Originally, the term "reaeration" evolved from early practices where the sludge in the final settling tank was not removed rapidly, so that compaction would allow lesser volumes to be returned. This required attempts to "re-activize" the sludge by reaeration. The detrimental effect of allowing oxygen deficiencies and the lack of knowledge of the magnitude of the reaeration time and oxygen quantities required, originally provided a poor process. The new concepts have allowed this modification to be very effective.

The most desirable alteration would be to have the sewage enter the aeration tank at various locations along the length of the aeration tank, or at various passes in the tank, always providing a portion at the head of the tank where the returned sludge is aerated or reaerated without the addition of appreciable quantities of primary effluent. This allows a speed-up of the process due to high concentrations of M.L.S.S. at the head of the tank with subsequent dilution by sewage addition, to levels amenable to proper final settling tank operation.

It may prove necessary to add all the sewage to a pass or location near the outlet end of the aerator and so practice the Sludge Reaeration Modification exactly, with approximately thirty to sixty minutes contact time between the conditioned sludge and the sewage. However, until the intensity of the overloading dictates, it would be preferable if possible, to practice Step Aeration with several points of sewage addition, to allow greater flexibility of operation, particularly of the contact times due to fluctuations of sewage flows and strengths. The inherent danger of fixed <u>contact tank</u> capacities suggests that some peak flows or sewage constituents may result in an existing contact time below that required for complete or adequate adsorption, particularly if the wastes at those times should consist of wastes containing appreciable <u>dissolved</u> B.O.D. (as opposed to the B.O.D. resulting from the colloidal or finely divided organic solids). Operating on The Step Aeration Modification allows greater flexibility.

In calculating the Sludge Age where the Step Aeration Modification is being practiced, the <u>total</u> weight of M.L.S.S. of <u>all passes</u> is required.

Usually, each plant will have a daily critical period. If at three o'clock each day, a sludge bulking condition manifests itself in the final settling tank, it is commonly the result of flow conditions and sludge qualities existing at the inlet end of the aerator at a time earlier by approximately the average flow-through time; that is, perhaps seven or eight o'clock in the morning. It would also require some time for the sludge to rise into view in the final clarifier. Therefore, to counteract the regular three o'clock bulking, alterations should be made to change the conditions normally occuring in the morning at the inlet end of the aerator. These are particularly the quantity and condition of the return sludge and the quantity and quality of the primary effluent flows.

The key is to <u>attempt</u> to keep the sludge in proper condition during all of the twenty-four hours of the day, seven days of each week, and to take advantage of all the off-peak periods to achieve this.

With the Step Aeration Modification, the point of application of sewage could be moved progressively nearer to the outlet end of the aerator as the last daylight peak period is approached. This results in greater quantities of the more voluminous sludge being accorded oxidation to proper conditions in the aerator during the off-peak and night hours. Flows at the start of the working day could then be accorded conventional treatment, with total sewage addition at the inlet end of the aerator, with the necessary larger quantities of acclimatized sludge for proper instantaneous F/M ratios, then being available. The intent can be seen to be the attempt to utilize these off-peak hours to provide oxidation of the sludge which has become less efficient during peak flow periods due to adsorption and synthesis.

During all these times, the rate of sludge return rates should be controlled to approach the required minimum rates as closely as time and staff will permit. And as previously stated, adequate dissolved oxygen must always be provided in all the aerobic portions of the plant's processes.

To achieve flexibility to allow these modifications to be practiced would require at least a means of sewage addition at one or more points along the length of the aerator. The necessary piping or channel would be required to be provided. The installation of lateral baffles may be required in the aeration tank to avoid short-circuiting.

A pipe could be installed within the tank to give the short contact period of the Sludge Reareation modification. Where parallel tanks are in use, the necessary improvement to plant effluents may accrue by directing a higher percentage of the sewage flows to one tank than to the other. If separate returns from each final tank is possible, a crossover of return lines would be desirable;

otherwise, a single return line with equal distribution to the aeration tanks and with unequal sewage distribution thereto, can still provide an improvement in effluent quality. A common effluent header from both aeration tanks will tend to equalize the flow rates to both final settling tanks.

Figures 2, 3 and 4 provide schematic suggestions of a few possible situations requiring only minor alterations to the plant structure to provide process modifications.

Where a distribution of Q is proposed, particularly in the mechanical aeration illustration, it would be desirable to provide control structures, preferably hydraulic, such as weirs, to cause all low flows to be treated via the conventional process. This would tend to avoid over-oxidation during low flow periods, particularly the night hours at some plants.

It should be noted that these alterations and proportioning of incoming flows are but suggestions to assist considerations at individual plants. Sewage flow rates and B.O.D. concentration variations at each will predicate the pertinence and propriety of these.

SUMMARY

It should be apparent that intelligent and ingenious operation of a plant that is receiving design or over-design flows can produce acceptable effluents and delay the date of necessary expenditures for plant expansions. Desirably, this should be recognized by the municipal officials.

This operation requires those same procedures as are required to operate a normally-loaded plant in the best possible manner, with further modifications to the plant process, as required. These modifications will commonly yield processes resembling the Step Aeration or Sludge Reaeration modifications and can sometimes be inaugurated with surprisingly few alterations to the existing plant. If the rate of over-loading continues to increase, even these process modifications and changes will prove to be inadequate and the effluent quality will deteriorate until plant expansions are inaugurated.

During this interim period of overloading that requires the best possible operation particularly, and to a similar degree during periods of normal loading, I have suggested that there are three necessary basic controls. These, in the order of their considered importance, are:

(1) maintaining at all times, adequate dissolved oxygen in all sections of the aerators, final settling tanks and return sludge lines. A minimum value of two ppm at the outlet end of the aerator is commonly suggested as effecting this result.

(2) removing the activated sludge from the final settling tanks, preferably at a rate at least equivalent to its rate of deposition there.

To calculate this minimum rate, a simple test and formula has been suggested here. The depth of sludge settling in the Thirty-Minute Settling Test is given the connotation of "b" and the depth of liquid above this being "a". The instantaneous required minimum rate of sludge return is suggested to be $(\frac{b}{a} \times 100)\%$ of the sewage flow to the plant; or more particularly, of the average rate of sewage flow during several hours immediately preceding the test.

(3) controlling the suspended solids concentration of the mixed liquor (M.L.S.S.) by maintaining either a proven adequate Sludge Age or F/M Ratio, calculated on the B.O.D. of the primary effluent. Sludge Age values of 3½ - 4 days have proven to be a proper value for many plants. The desired Sludge Age therefore dictates the necessary rate of sludge wasting which usually should be performed on a continuous basis.

FIG. 2

MINOR ALTERATIONS FOR PROCESS MODIFICATION

(SINGLE TANKS)

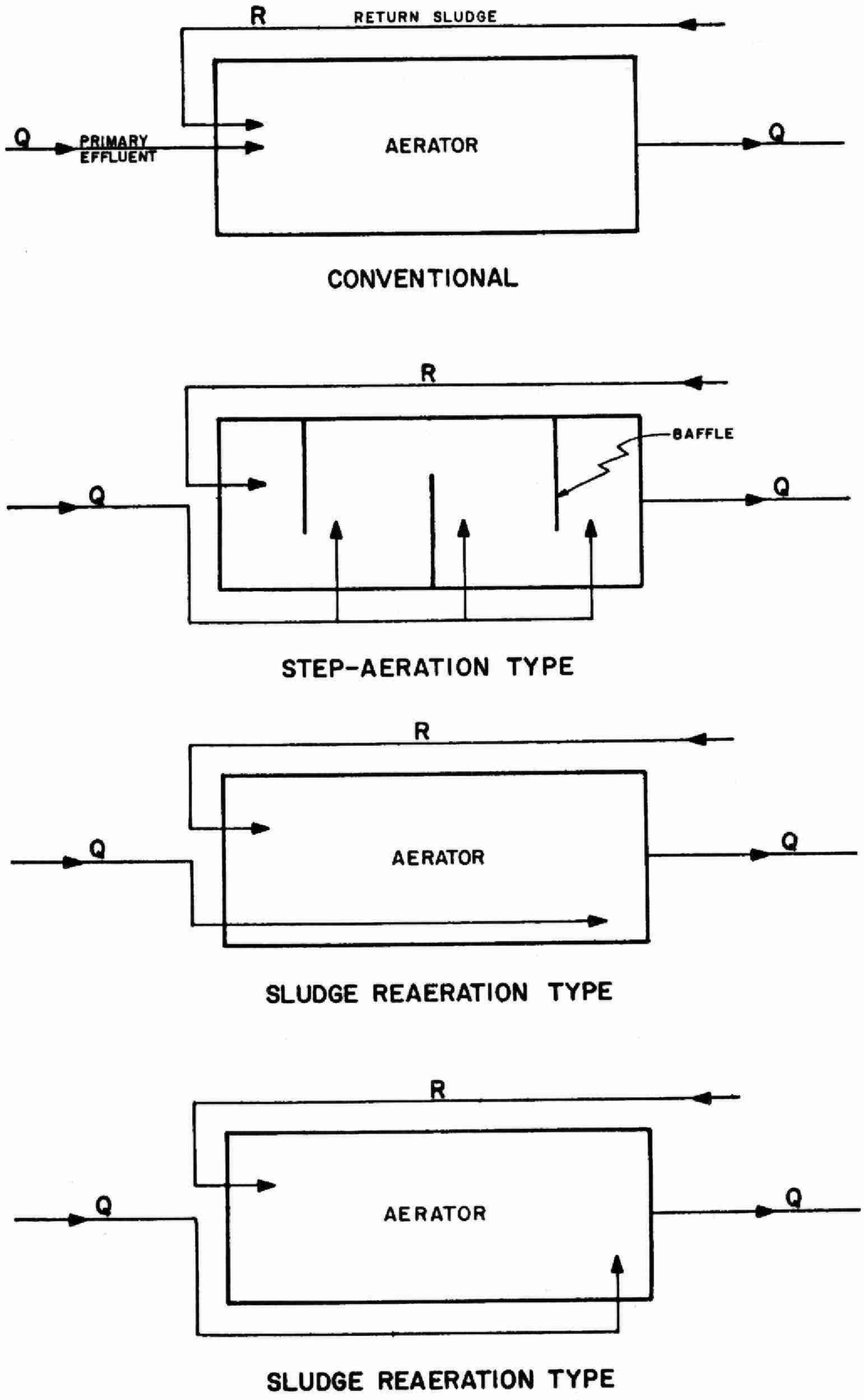
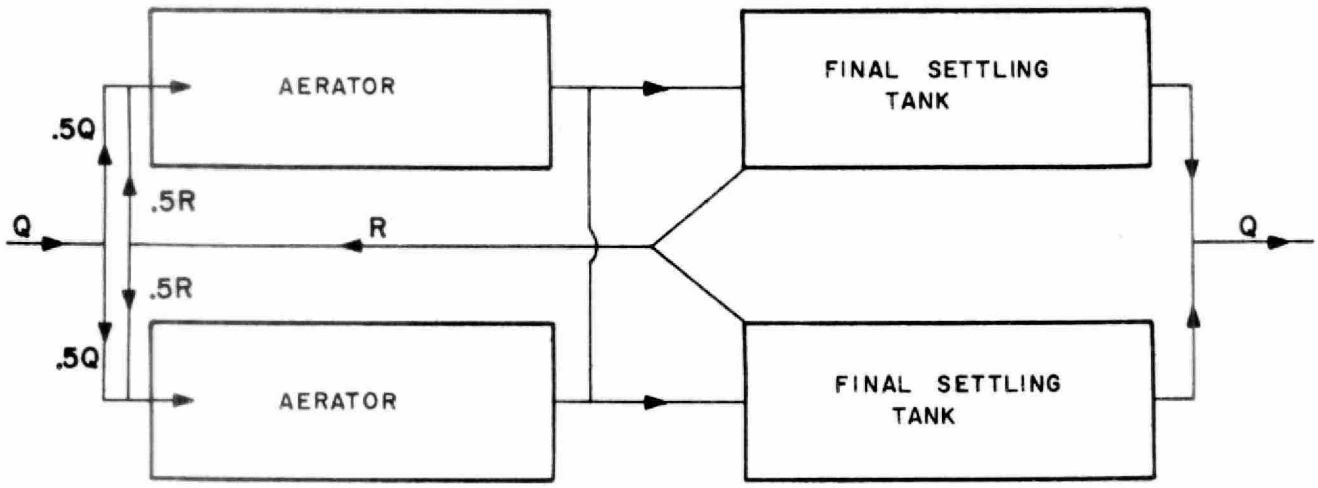
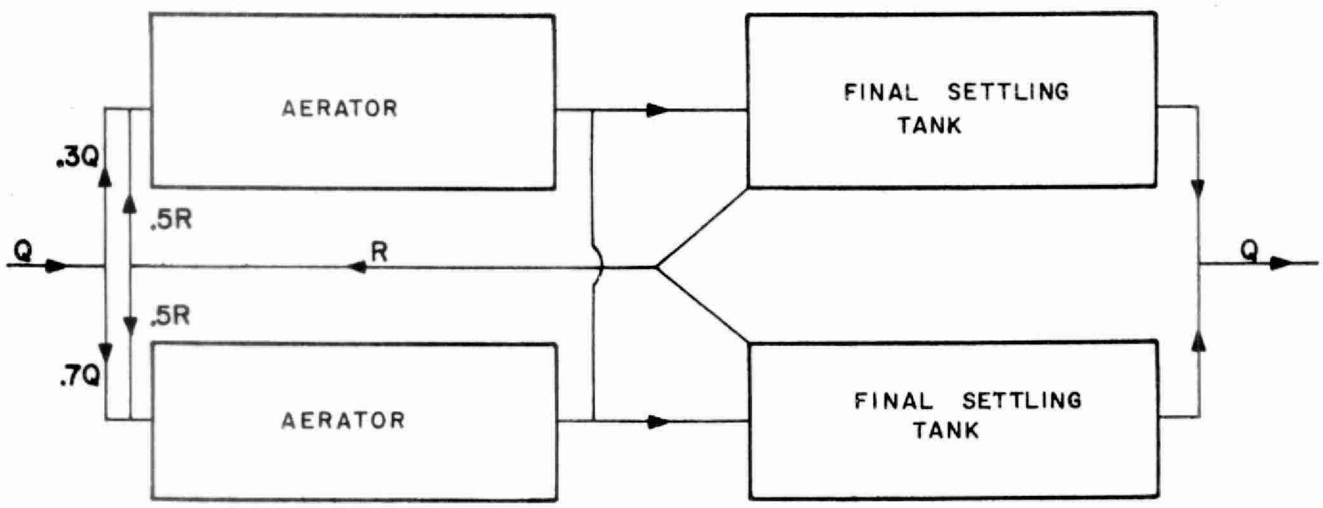


FIG. 3

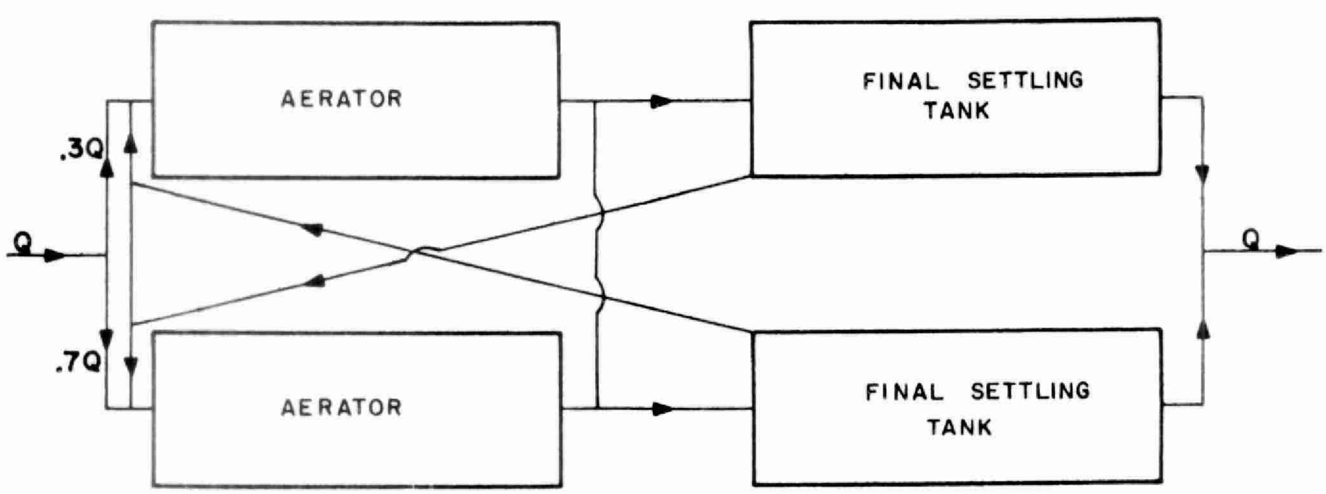
MINOR ALTERATIONS FOR PROCESS MODIFICATION
(DUAL TANKS)



CONVENTIONAL (SINGLE SLUDGE RETURN LINE)



MODIFIED (SINGLE SLUDGE RETURN LINE)

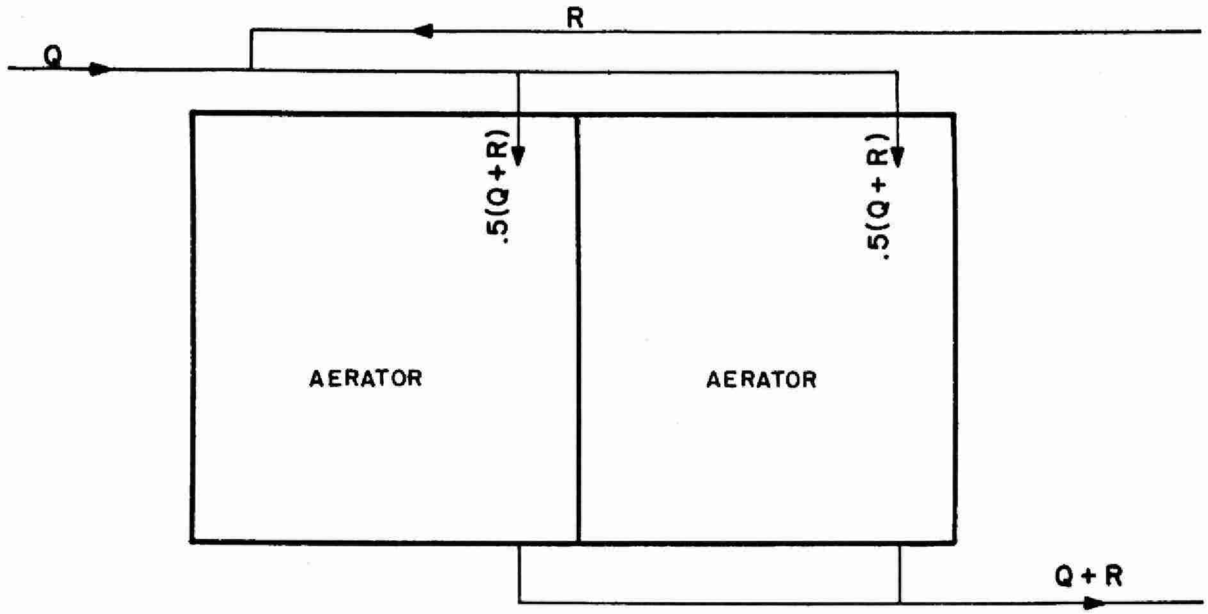


MODIFIED (DUAL SLUDGE RETURN LINE)

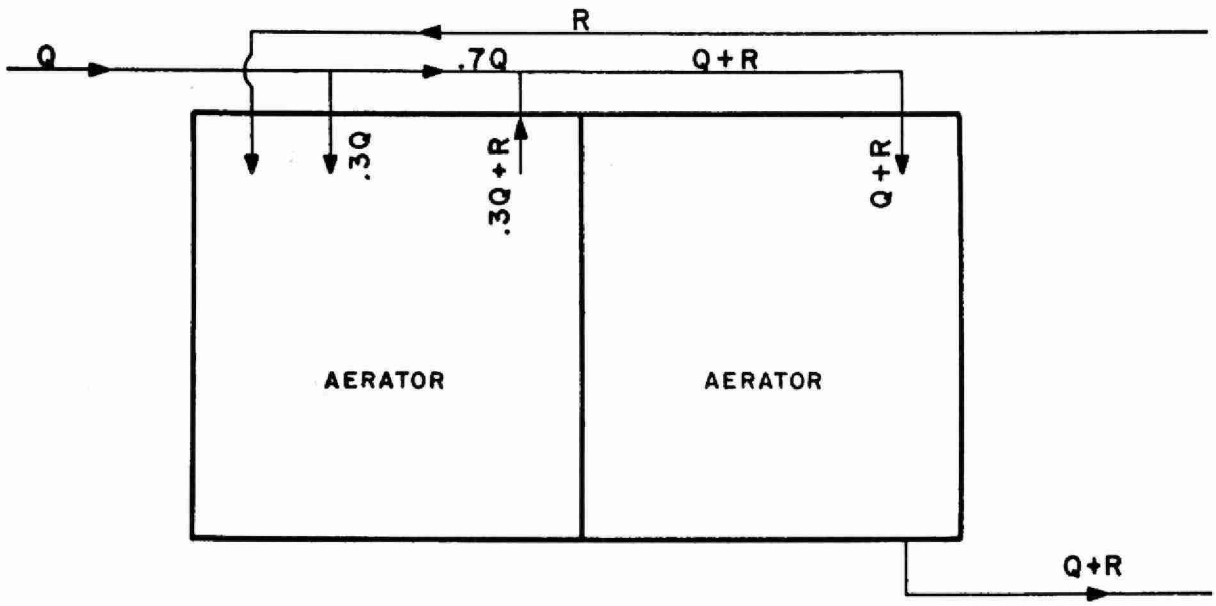
FIG. 4

MINOR ALTERATIONS FOR PROCESS MODIFICATION

(MECHANICAL AERATION – DUAL TANKS)



CONVENTIONAL



STEP–AERATION TYPE

MINIMIZING THE EFFECTS OF SHOCK LOADS

by

D. McTavish

Regional Assistant Supervisor
Division of Plant Operations - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 23, 1963

MINIMIZING THE EFFECTS OF SHOCK LOADS

by

D. McTAVISH

Regional Assistant Supervisor

INTRODUCTION

"Shock" denotes surprise -- something other than normal. The dictionary defines shock as "sudden and disturbing physical impression or injury inflicted on stability". Shock, therefore, implies more than normal change or variation. It implies change beyond the normal -- a change which affects stability.

Those in the water pollution control field know that change and variation occur continuously. The rate of flow changes, the concentration of BOD and SS vary and all other characteristics attributal to sewage are constantly changing.

Shock loads are concentrations, temperatures, volumes and/or masses of materials which exceed the limits of normal variation for which a treatment works was designed and which inflict injury on the stability of that system. These limits may be exceeded positively or negatively. A pH below the lower tolerance limit or above the upper tolerance level will injuriously affect the process. Some materials are only of concern if they exceed a lower or upper limit. For example, nitrogen concentrations below 1/20th the BOD concentration will cause concern as will metallic ions above 5 ppm concentrations. However, higher concentrations of nitrogen and lower concentrations of metallic ions will not be a cause for concern. Variables such as rate of flow will cause operating problems if they are in excess of an upper or less than a lower limit.

Graph one indicates the normal variation and the greater than normal variations (shock loads) which exceed the limits of the process. It should be noted that each variable has its own upper and lower limit and that these limits also depend on time. A BOD concentration of 1,000 ppm may be tolerated for 5 minutes but not for 5 days in a plant designed for sewage having a concentration of 150 ppm.

Knowledge of the various limits of your process are important if shock loads are to be dealt with effectively. If trends are noted, knowledge of the process limits will allow time for advanced planning. If cyclic loading problems occur, such as high flows in the spring or high BOD concentrations during the canning season, it may be possible to deal with them effectively if knowledge of the limits of your process is available.

NORMAL VARIATIONS

We are well aware that variations occur in sewage with respect to all its characteristics. This is expected and our plants are designed to deal with variations. However, we sometimes do not have an appreciation of the magnitude of "normal" variation. Without some knowledge of this, it is most difficult to note an occurrence which is not normal. Often we pay no attention to a certain characteristic of the sewage, but when operating problems occur, we study this characteristic, see that it varies considerably and come to the conclusion that this is the characteristic or variable in the sewage that is causing the problem. It may be, however, that this particular characteristic has always varied as it now does and, in fact, this is only normal variation.

The variations experienced with the various characteristics of sewage will be different from plant to plant and each plant should be reviewed to discover its own particular sewage variations. Plants which receive continuously a certain metallic ion can become acclimatized to that ion and function quite well with concentrations which would constitute a shock load to another plant not experiencing a continuous feed of ion.

Therefore, the following discussion of "normal variations" should be regarded as a guide rather than a precise description of your own plant's sewage.

1. Flows

The flow of sewage to a plant varies hourly, daily, and seasonally. The smaller the community and/or the older the sewage collection system, the greater will be the variations. Combined sewers will greatly influence the variations.

Generally speaking, the flow will reach a peak during the day and become a minimum after midnight. Flow rates during the week days will be greater than on weekends and flows experienced during the spring months usually exceed those of other seasons.

The hourly variations in flow rate at a small plant may vary from 50 to 200 percent of the average daily flow rate. Graph two indicates the average flow rate for four hour periods over one week's time at an OWRC plant in a medium sized city. The greatest difference in flow rate occurred on Monday as a result of rain. The minimum daily flows occurred on Saturday and Sunday. The peak hourly flow rates occurred from 8 a.m. to 8 p.m. with the absolute maximum occurring at 1 p.m.

The flow rates varied generally from 80 to 120 percent of the average with the exception of Monday when the variation was from 60 to 160 percent of average. Variations in flow at small plants would normally be greater than those indicated on Graph two. (50 to 200 percent)

The same graph shows the concentration of BOD in the final effluent during each 4 hour period. The concentrations fluctuate but a pattern is not evident. The concentration was higher, however, during the heavy flows of Monday.

2. BOD

In general, the variation in concentration of BOD follows the pattern set for flows. The highest concentrations are experienced during the mid-day with the lowest concentration occurring during the early morning hours. The fluctuations in concentrations, expressed as a percent, will be approximately the same as those for flow, i.e. from 50 to 200 percent of the average. This generalization may not be true for communities with a high percent of industrial wastes.

The fluctuations in concentration are lessened considerably by primary treatment.

Graph three depicts the variation in concentrations from the average for raw sewage and primary effluent from a medium sized Ontario city. Each point on the graph represents a four hour composite sample. It can be noted that both sewages "peaked" during the day with the primary effluent lagging from 2 to 4 hours behind the raw sewage.

The raw sewage increased as much as 120 ppm above the average and decreased to the same extent below average. The primary effluent did not vary as greatly. The highest concentration was 60 ppm above average and the lowest 80 ppm below average.

The loading of aeration tanks is normally involved with pounds of BOD rather than concentration of BOD. Pounds of BOD combines the flow and concentration. It is, therefore, evident that, since the flow rate and BOD concentrations experience peaks and valleys approximately the same time as one

another, the resulting variation in rate of feed (pounds of BOD per day) will be greater than either the flow or concentration.

Graph four depicts the variation in feed from the average for one week at the same plant referred to previously. The variations are shown for primary effluent and raw sewage and are expressed as a percent of the average. The feed rate of the raw sewage varied from 10 to 220 percent of average. The feed rate of the primary effluent varied from 10 to 200 percent of the average.

3. pH

pH affects the rate of enzyme reactions; some bacteria have high rates of reaction at relatively low pH values while others have high rates of reactions at high pH values. The optimum pH ranges for activated sludge treating domestic waste is from 7.0 - 7.5 with an effective process range from 6.0 - 9.0. At a pH of 4.0, the process will be only 43 percent effective and at a pH of 10, only 54 percent effective. A rapid change in pH may decrease the respiratory activity as much as 75 percent.

4. Toxicity

Many substances exert a toxic effect on the biological oxidation process. Partial or complete inhibitation may result, depending on the substance and its concentration. The concentration of a substance which will exert a toxic effect is also influenced by other factors such as food, concentration, temperature and the nature of the organism. In some cases, the biological population will acclimatize itself to a concentration level of a toxic substance. It has been noted that, in the presence of non-acclimatized sludge, sulphide completely inhibited biological growth in concentrations in excess of 25 ppm. Adaption for 24 - 48 hours increased the tolerance level to 100 ppm. In the case of heavy metals such as copper, zinc and chromium adaption for 72 hours increased the tolerance level from 1 - 5 ppm to greater than 75 ppm.

Suggested maximum safe levels for various toxic materials are given on Page 79 of the Intermediate Sewage Works Course given from March 5th to 9th, 1962.

5. Temperature

The rate of enzyme reaction has been shown to approximately double with each 10° C. rise in temperature up to 35°C. Temperature also has an effect on sedimentation. As the temperature increases, the viscosity decreases and sedimentation is improved.

The temperature of the sewage is usually a few degrees above the temperature of the water supply. In general, the raw sewage temperatures average from 40 to 55°F. in the winter and from 55 to 70°F. during the summer. A temperature rise of from 7 to 12°F. will be experienced in the summer months by sewage flowing through an activated sludge plant with a similar fall in temperature during the winter months.

SOME EFFECTS OF SHOCK LOADS

Most of the effects of shock loads are quite well known to operators and have been described in the Intermediate Sewage Works Course. However, some of the effects are listed below:

1. Flows

Large flows (flows above the design capacity) may increase the aeration tank loading and will shorten detention time. This, coupled with shorter detention time in the final clarifiers, can result in poor settling and the loss of solids from the system. The time of recovery of the system will depend on the extent of the loss of solids.

Low flows can result in sedimentation of sludge in sewers, inlet channels and wet wells. This can lead to septic sewage and odours.

2. BOD

High concentrations of BOD may result in rapid sludge growth and bulking sludge. The dissolved oxygen concentration may decrease if remedial action is not taken and if it falls below 0.2 ppm to 0.5 ppm, the system becomes oxygen dependent and the rate of BOD removal is decreased.

Low concentrations can result in underloading, nitrification and the inefficient use of air. BOD removal is usually high in this case, but larger suspended solids concentrations in the effluent will result.

3. pH

Extreme variations in pH will result in decreased biological activity and overloading may occur. As a result, bulking sludge will be experienced in the final clarifier. Graph five shows an OWRC plant which experiences shock loading with respect to pH.

It should be noted that the activated sludge life tends to buffer the mixed liquor and, as a result, the pH variations in the mixed liquor will not be as great as those of the raw sewage. The Ph of the mixed liquor should be maintained between 6.0 and 9.0

4. Toxicity

High concentrations of toxic materials can inhibit biological life in aeration sections and in digesters. Bulking sludge may result in the aeration section and poor quality digestion and high volatile acid concentrations may occur in the digester.

5. Miscellaneous

Shock loads of materials such as feathers may result in blocked bar screens and cause other physical defects. Deficiencies in nitrogen can bring about a bulking and a poor quality activated sludge. Substances such as phenols may increase the oxygen demand and produce an unsightly effluent.

MINIMIZING THE EFFECT OF SHOCK LOADS

1. Flows

Combined sewer systems and those greatly affected by infiltration will periodically supply very high flows to treatment plants. Usually these flows occur over a short period of time.

Often complete treatment can be given to flows of this type if storm tanks are available. Some plants are designed with storm tanks and their use is quite evident to the operator. Other plants may not be provided with storm tanks but it is often possible to improvise. If the plant is normally operating without all units in service, then it is often possible to utilize these units as storm tanks. Portable pumps and chlorine may also be required if storm flows are dealt with in this manner.

Without storage being available it is advisable to give as much treatment as possible to the flow without destroying the process. The aeration section is usually the most critical part of the process. During high flows care should be taken to keep the hydraulic (and organic) load within the process limits of the aeration section. This will probably entail complete primary treatment, partial secondary treatment and chlorination of the entire effluent.

If the plant is subjected to heavy flows and solids are lost in the effluent, the formation of a good activated

sludge can be hastened by the addition of ferric chloride. The ferric chloride aids in coagulation and a good activated sludge requires a small concentration of iron and aluminum salts to aid in good settling. Care must be taken in the addition of ferric chloride not to affect the pH adversely. We have, at the suggestion of the Purifications Branch, used this method with success to obtain a good activated sludge at two of our plants that have suffered from storm flows.

Many new plants are subjected to flows much below the plant design flow. This results in extended aeration times, nitrification and pin-point floc, and the use of a large volume of air for each pound of BOD removed. If possible, it is desirable in this instance to place part of the plant in operation. If the plant has good flexibility, this is often not too difficult to accomplish. The use of only one clarifier and partial by-pass of this clarifier will also help increase the load on an aeration section if it is not possible to lessen the aeration tank volume. The minimum concentration of mixed liquor solids consistent with the limits of your plant should also be carried to help obtain a better feed to micro-organism ratio.

Low flows may result in long detention times in wet wells which were designed for future flows. Every effort should be made to keep the detention in the wet wells as low as possible. This may be accomplished by having the pumps running over the minimum change in level in the wet well or by using only part of a wet well. Chlorine may also be required in wet wells which provide extended detention times.

2. BOD

Sudden and large changes in the BOD loading of a conventional activated sludge plant can often not be dealt with effectively by the plant. This knowledge has led to the development of several modifications to the conventional process. Most of these modifications have been discussed in previous courses but bear repeating.

The average loading ratios employed at larger plants are from 0.3 to 0.4 pounds of BOD per pound of mixed liquor suspended solids. This range is more likely to be 0.2 to 0.3 pounds of BOD per pound of MLSS for small plants. Due to air requirements, ability of sludge to separate, return sludge pumping problems and other design restrictions, the conventional process is usually also limited to loadings of from 25 to 30 pounds of BOD per day per 1000 cubic feet of aeration tank volume. Some of the disadvantages of the conventional activated sludge process are listed below:

(i) BOD loadings are limited to about 35 pounds of BOD per day per 1000 cubic feet of aeration tank volume.

(ii) High initial oxygen demand of mixed liquors is encountered.

(iii) There is a tendency to produce bulking sludge.

(iv) High sludge recirculation ratios are required for wastes with high BOD concentrations.

(v) High solids loadings are imposed on final clarifiers.

(vi) High air requirements per pound of BOD removed.

All of the modifications to the conventional process have evolved in an attempt to eliminate or minimize some or all of the above.

MODIFICATIONS

Step Aeration

The process provides for reaeration of the returned sludge which allows an opportunity for the oxidation of absorbed organic matter and the subsequent feeding of sewage in a stepwise manner, (see Graph 6). In order to be effective, multiple pass tanks are required. The system depends on maintaining an activated sludge with high absorptive properties so that organic pollutants can be removed with a relatively short contact period. One of the main advantages of the process is that it controls the solids loading on the final clarifiers. It allows higher BOD loadings and shorter detention times and will produce an activated sludge with good settling properties.

Biosorption Process

This process was developed in Austin, Texas and is similar to step aeration (See Graph 7). It employs a system of reaeration of sludge to restore its purification capacity and a very short 20 to 30 minute aeration period with unsettled sewage. The return sludge volume is equal to about 40 percent of the sewage flow and is kept under aeration from 5 to 7 hours before mixing with raw sewage.

Kraus Process

The Peoria, Illinois sewage plant was subjected to heavy organic loadings from industrial sources and bulking sludge resulted. The Kraus Process was then developed at the plant to control the sludge index (See Graph 8). The process involves separate aeration of a mixture of digestion tank supernatant, digested sludges, and some activated sludge to produce a well

nitrified mixture. This mixture added to the return sludge adds weight to the activated sludge and serves as a means of controlling the index. The nitrates act as a reserve supply of oxygen to help and maintain aerobic conditions under heavy organic loads.

Contact Stabilization

This process is similar to step aeration except aeration of the return sludge is completely separated from the primary effluent. The return sludge is aerated for from 5 to 7 hours and is mixed in a separate tank with primary effluent. The mixed liquor is aerated for approximately 30 minutes.

The BOD/SS ratio is not as critical with this modification as compared to the conventional process. The sewage plant operator can often, with some ingenuity, convert his conventional process to a form of one of the above modifications.

In many cases, the plant can deal with large loads if informed. Small communities in particular require close liaison between the plant operator and the industry causing the problem. Advanced knowledge will provide time to increase aeration solids, increase air supply, prepare chlorination equipment and, in general, give the plant a "fighting chance". An example of this was reported by the treatment plant at Delta, Ohio (population 2,400).

The plant was notified that a large volume of creamery waste had been dumped. The sludge return rate was increased as was the air supply. Sampling later revealed a grab sample of 37,000 mg/l, BOD with 8 hour composites of 1,472 mg/l, 486 mg/l and 2.9 mg/l for the raw, primary effluent and final effluent respectively. Raw sewage was directed periodically to the aeration section without primary treatment. At the same time, return sludge was wasted at a rate of 30%. The purpose was to remove the light sludge produced by the milk and replace it as fast as possible with more dense solids from the raw sewage.

3. Toxicity

Often shock loads of toxic material are noted only after they have caused the problem. However, if the toxic load involves the ions of heavy metals, the problem will probably appear in the aeration section and later in the digestion system. It is, therefore, important to observe the operation of a digester rather carefully after an upset in the aeration tank has occurred. Digester problems can be minimized if detected in time and the loading of the digester postponed until it and the raw sludge quality return to normal.

Activated sludge which has been adversely affected by toxic materials will often recover with time. However, it is sometimes advisable to remove this sludge and rebuild new sludge.

Sampling of the sewage for toxic materials should be commenced as soon as trouble is noted in an effort to locate the source of the problem. Often the operator will suspect one or two industries due to his knowledge of the local industries. The industry should be contacted and informed of the problem at the plant and their cooperation in eliminating the problem solicited.

4. pH

Shock loads of pH can often be detected immediately by the plant. Equalizing basins or storage in storm tanks with subsequent feeding to the flow at a slow rate can be employed. However, municipal officials should be contacted concerning pH problems as this will also be detrimental to the sewer system as well as the plant. Unlike some of the toxic ions, the pH shock loads do not usually affect the digestion process.

SUMMARY

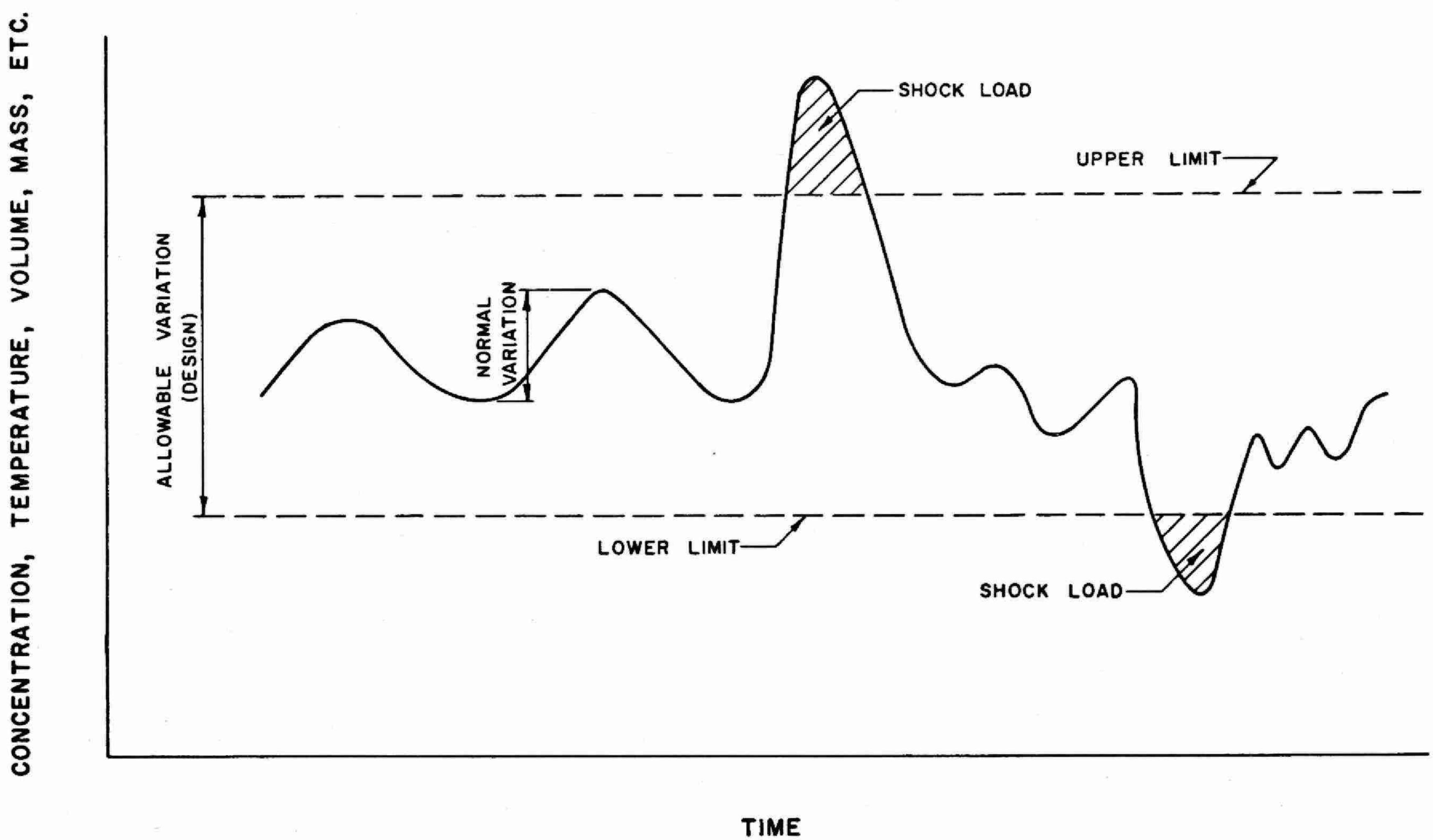
The operator should know and use the design limits and capabilities of his treatment plant to their fullest in order to effectively deal with shock loads. Problems with shock loads should be reported to municipal and other officials involved with the system. In order that an operator be able to intelligently recognize shock loads, he should be aware of the normal variation in the characteristics of the sewage as well as the design capabilities of the process.

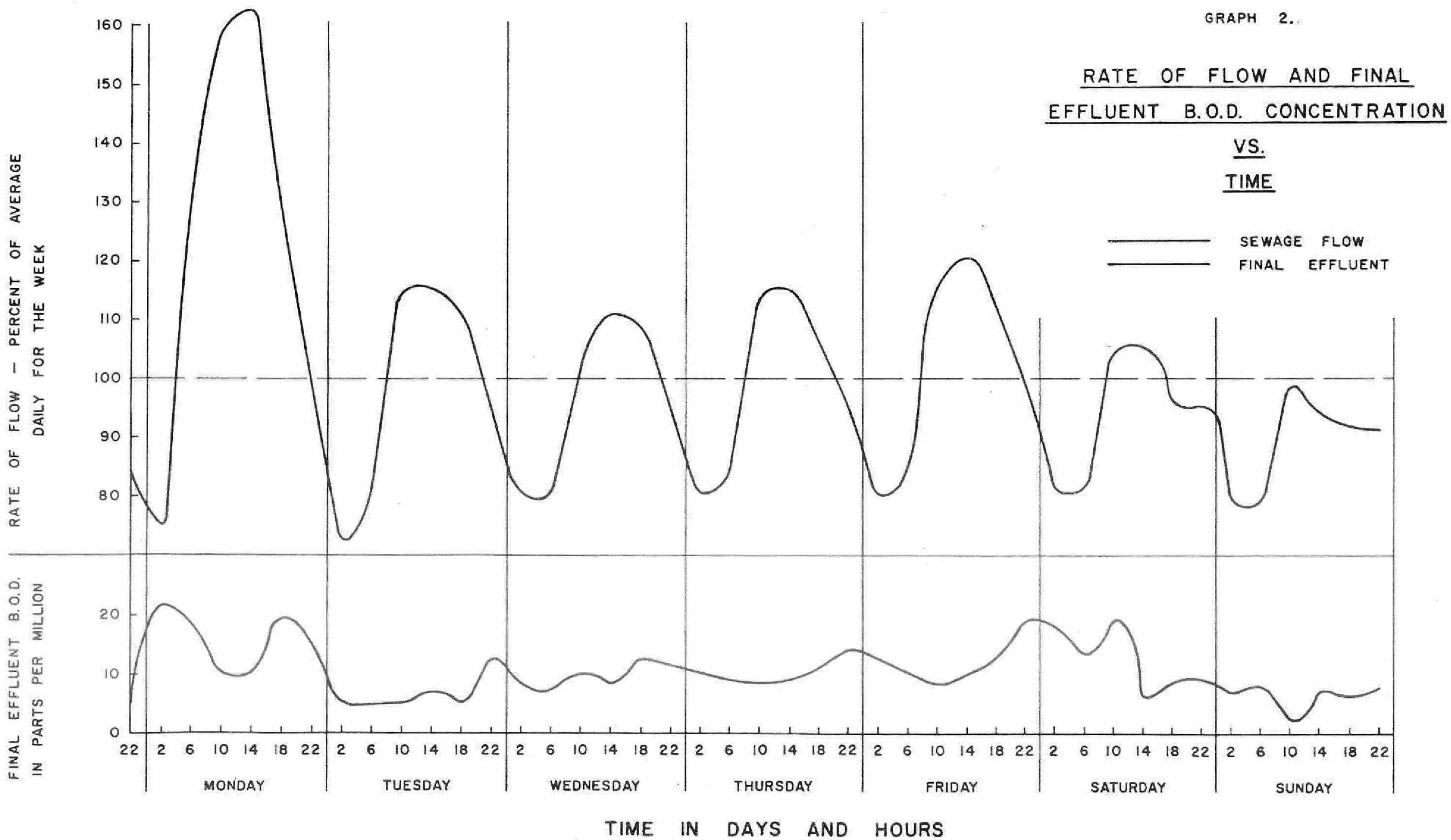
Many of the shock loads with respect to flow, BOD, nitrogen deficiencies etc. can be dealt with by converting the conventional process to one of the modifications described earlier. This may be necessary for short term operation until the source of the shock load is removed or until the plant is enlarged. It may also be that the modification is the most effective and economical way of dealing with the problem on a long term basis.

Shock loads resulting from toxic ions, pH changes and/or extreme temperatures usually require correction at the source. Sampling and reporting of the problem to the proper officials is most important.

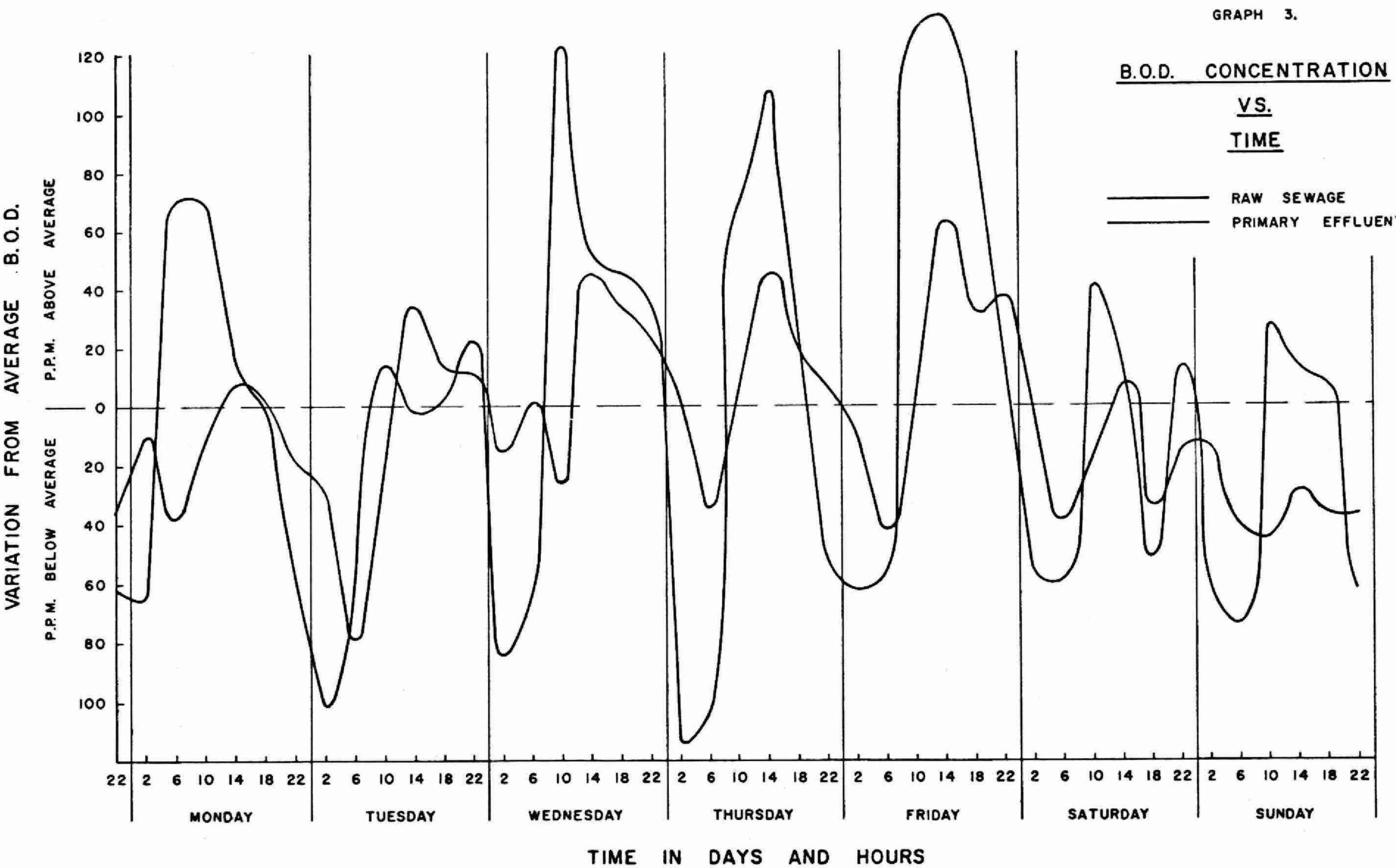
GRAPH 1.

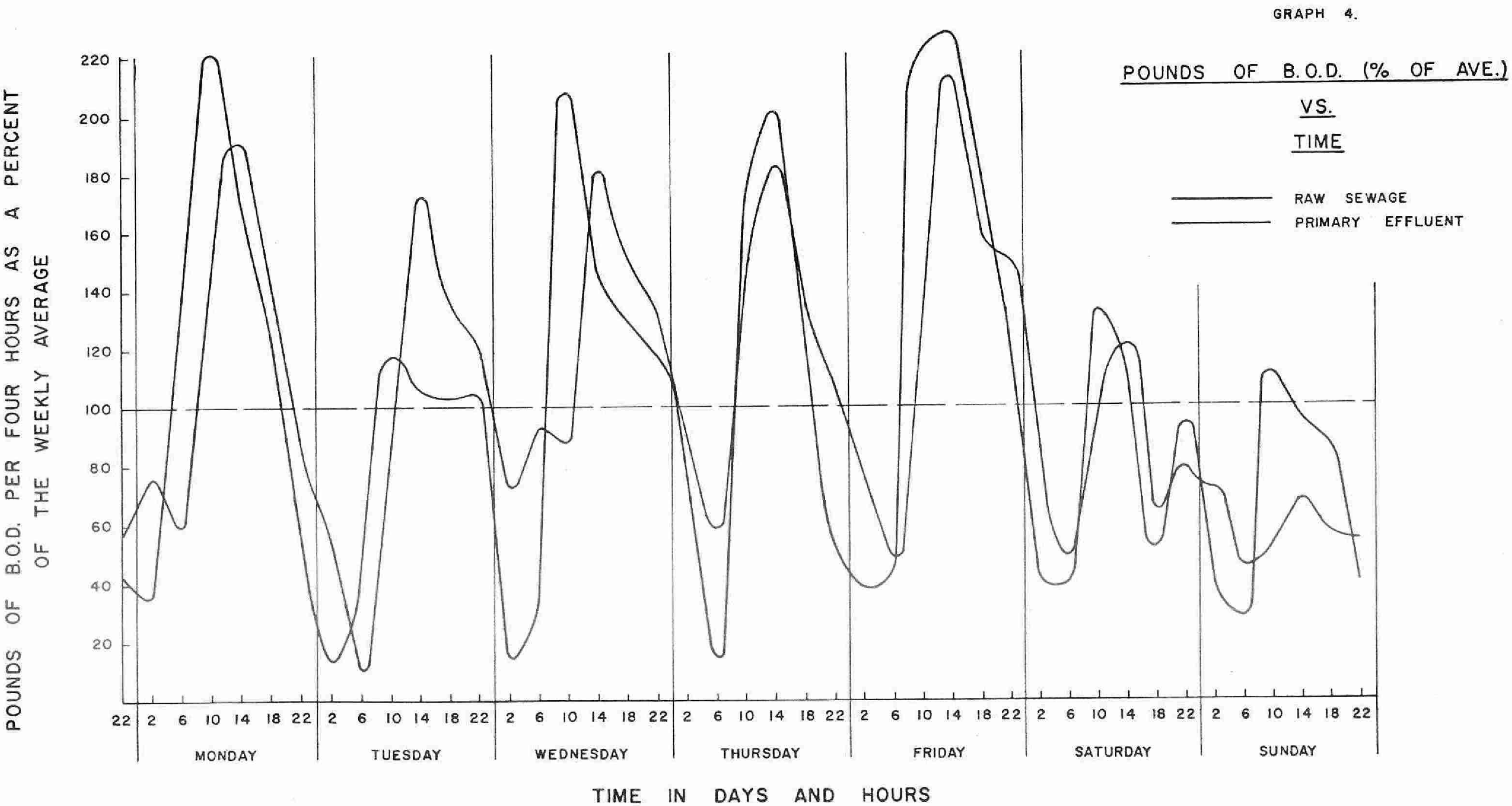
SHOCK LOADS IN THE ACTIVATED SLUDGE PROCESS



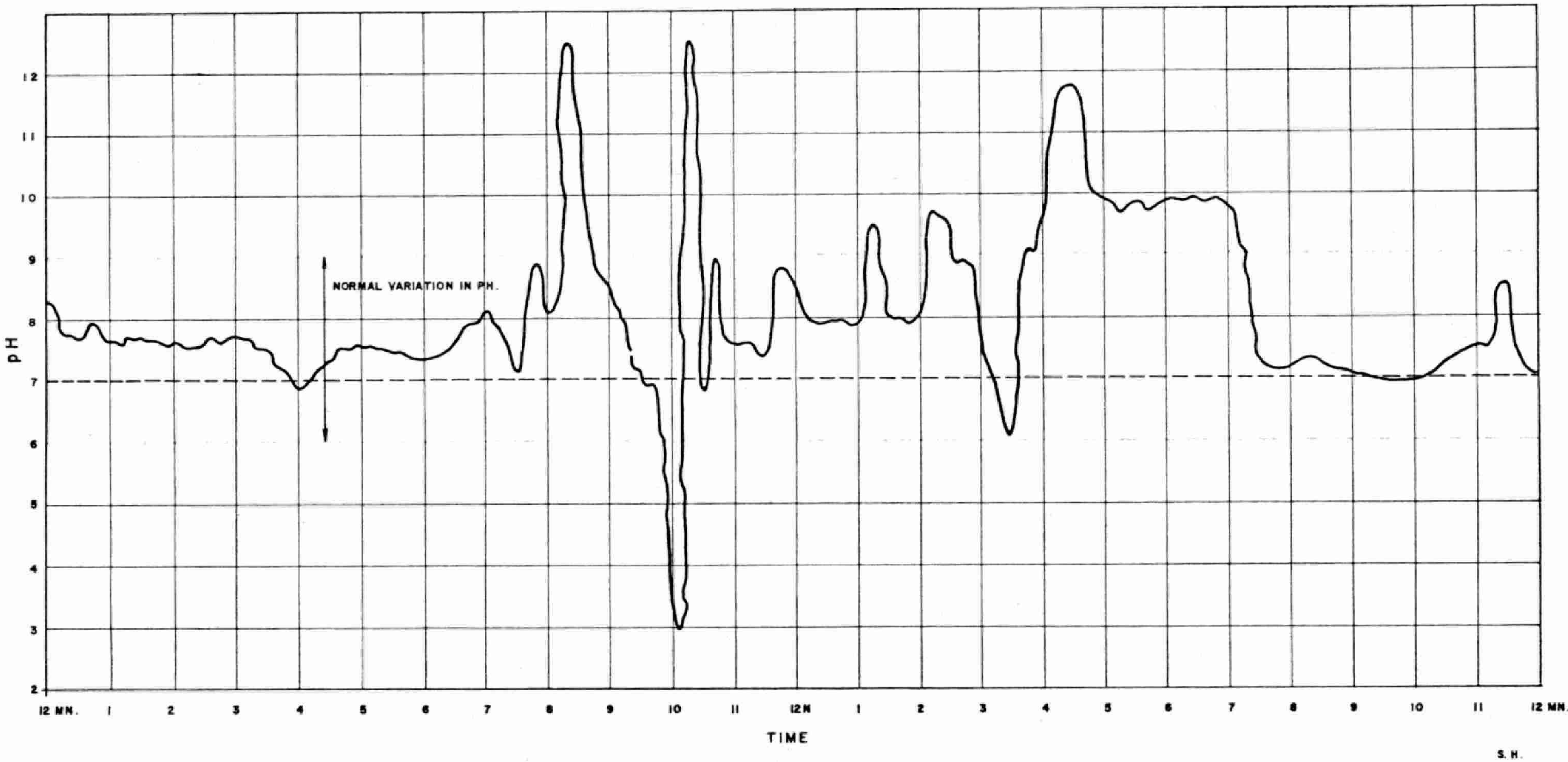


GRAPH 2.
RATE OF FLOW AND FINAL EFFLUENT B.O.D. CONCENTRATION VS. TIME
SEWAGE FLOW
FINAL EFFLUENT
RATE OF FLOW – PERCENT OF AVERAGE DAILY FOR THE WEEK
FINAL EFFLUENT B.O.D. IN PARTS PER MILLION
160
150
140
130
120
110
100
90
80
20
10
0
22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
TIME IN DAYS AND HOURS


GRAPH 3.
B.O.D. CONCENTRATION
VS.
TIME
RAW SEWAGE
PRIMARY EFFLUENT
VARIATION FROM AVERAGE B.O.D.
P.P.M. ABOVE AVERAGE
P.P.M. BELOW AVERAGE
120
100
80
60
40
20
0
20
40
60
80
100
22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
TIME IN DAYS AND HOURS


GRAPH 4.
POUNDS OF B.O.D. (% OF AVE.)
VS.
TIME
RAW SEWAGE
PRIMARY EFFLUENT
POUNDS OF B.O.D. PER FOUR HOURS AS A PERCENT OF THE WEEKLY AVERAGE
220
200
180
160
140
120
100
80
60
40
20
22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22 2 6 10 14 18 22
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
TIME IN DAYS AND HOURS

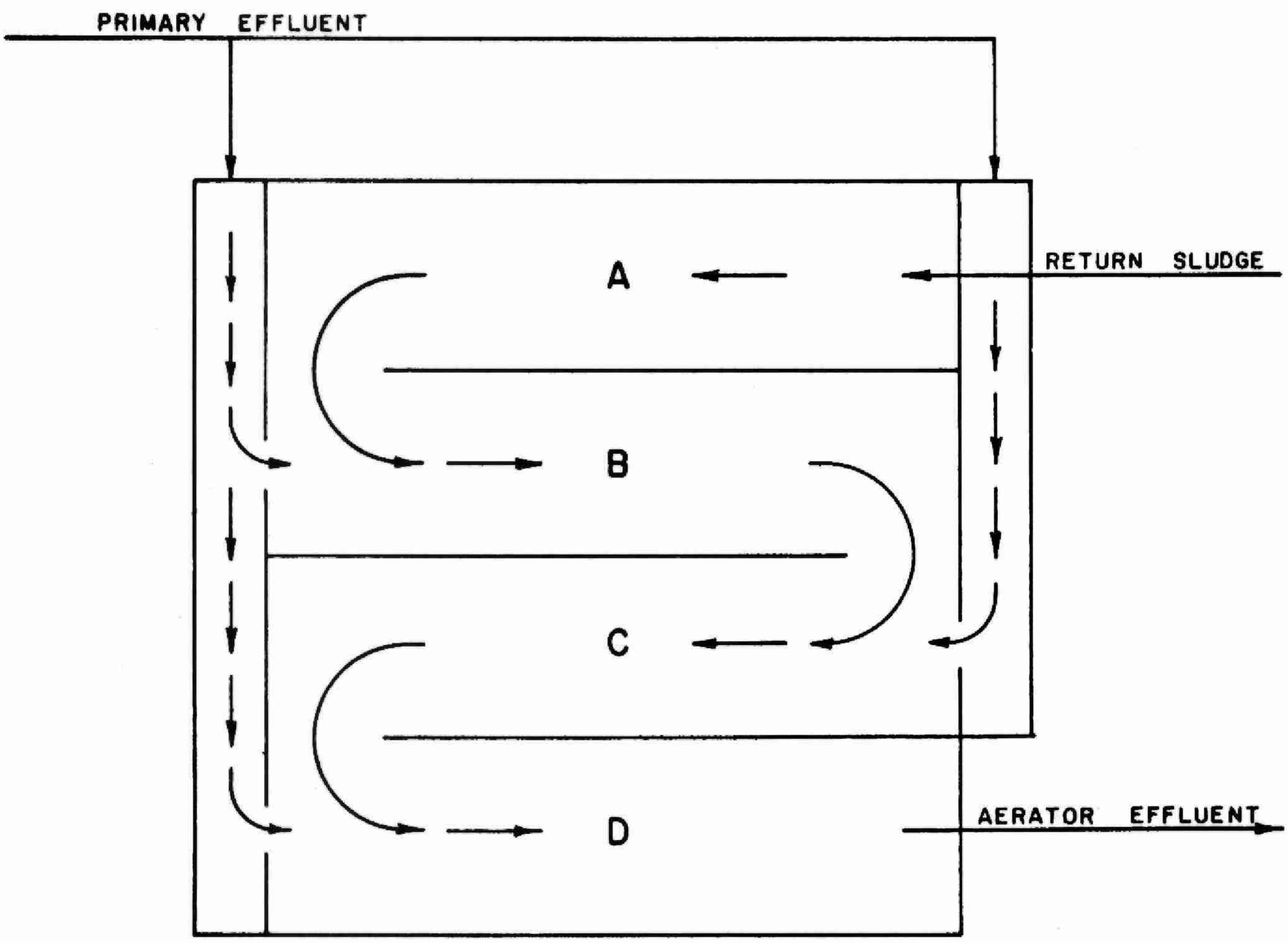
GRAPH 5.
HOURLY VARIATION IN pH



GRAPH 6.

MODIFICATIONS TO THE ACTIVATED SLUDGE PROCESS

I STEP AERATION

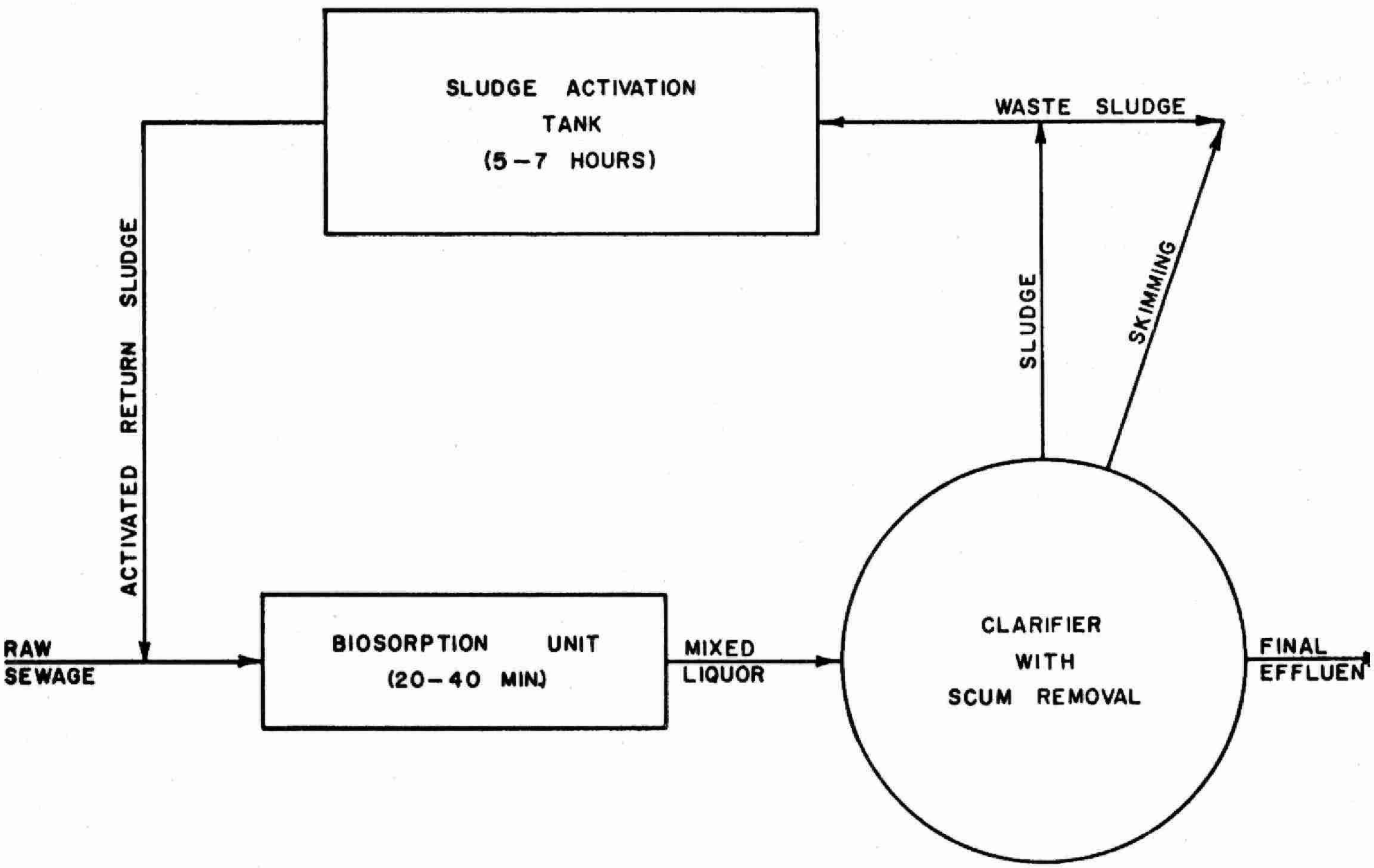


STEP AERATION PROCESS EMPLOYING
A FOUR PASS TANK WITH ADDITION
OF SEWAGE IN B, C & D PASSES

GRAPH 7.

MODIFICATIONS TO THE ACTIVATED SLUDGE PROCESS

II BIOSORPTION PROCESS

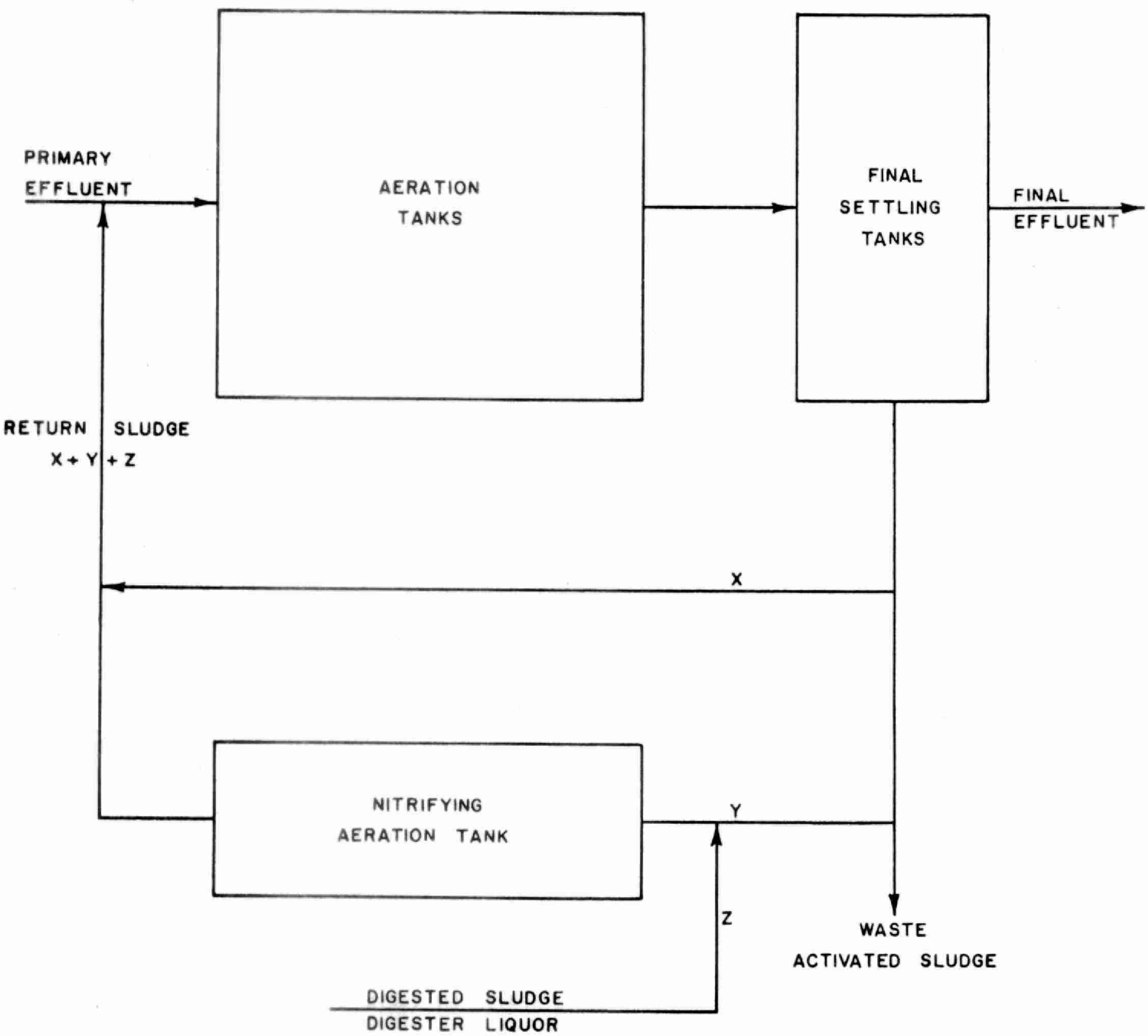


SCHEMATIC DIAGRAM OF BIOSORPTION PROCESS

GRAPH 8.

MODIFICATIONS TO THE ACTIVATED SLUDGE PROCESS

III KRAUS PROCESS



DEWATERING OF SLUDGE ON SAND BEDS

by

W. A. Steggles

Supervisor, Stream Sanitation - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 23, 1963

DEWATERING OF SLUDGE ON SAND BEDS

by

W. A. STEGGLES

Supervisor, Stream Sanitation

USE TODAY

Sand beds used for sludge dewatering before final disposal remain a commonly used device. Normally employed in the treatment of digested sludges, there are some applications using raw sludge, though these are few. The following summary illustrates the current use of sand beds at municipal sewage treatment plants in Ontario.

Population group served	total number of plants	number with sludge beds	Percentage using beds
less than 1,000	55	1	2
1,000 - 5,000	143	23	16
5,000 - 10,000	37	14	38
10,000 - 25,000	22	6	27
25,000 - 50,000	13	1	8
50,000 - 100,000	7	2	29
100,000 - plus	5	0	0
	282	47	17

About one-sixth of the plants now in operation use sludge drying beds. This probably reflects the development of vacuum filtration and the increased use of liquid hauling, however, the above figures are unduly weighted by the number of municipal septic tank systems still in use in the province. Seventy-five per cent of the treatment units serving populations less than 1000 persons, and nearly one-half (48 per cent) of the plants serving populations ranging from 1000 to 5000 persons are comprised of generally inefficient sedimentation units or large septic tanks. Sand beds are widely used for sludge dewatering, especially at medium sized plants which serve populations of 5000 to 10,000 persons.

Ample land and a demand for dewatered sludge for soil-conditioning can make the process attractive in smaller communities.

A similar inventory of plants approved in Ontario since 1956, suggests that the extent of use of sand beds for sludge dewatering has not changed appreciably. Covered sand beds have not been proposed during this period.

Population group	plants approved since 1956	plants with sand beds approved	percentage with beds
less than 1,000	38	0	0
1,000 - 5,000	45	12	27
5,000 - 10,000	16	4	25
10,000 - 25,000	19	6	31
25,000 - 50,000	8	1	12
50,000 - 100,000	4	0	0
100,000 - plus	2	0	0
	132	23	17

PROCESS OF DEWATERING - THEORY

Both drainage and evaporation operate in the drying process to reduce the volume of the wet sludge by 50 - 75 per cent. Thereby, the solids content of the sludge is increased from about 6 - 10 per cent to about 40 per cent.

In removing well digested sludges from the pressurized environment of the digester, dissolved gases in the sludge are released, which tend to float the sludge particles and facilitate dewatering by drainage. This so-called "gassing" and draining period is predominant during the first day following application to the beds. Thereafter evaporation continues at a slower rate and depends on the bed temperature and the circulation of air above the bed. Drainage is responsible for greater removals of moisture than evaporation.

Continued loss of moisture causes horizontal shrinking and surface cracking of the sludge cake which in turn promotes evaporation from the increased surface area.

Poorly - digested sludge dewaters less readily on beds than well - digested sludge. A dense mat is formed which is of course difficult to separate from the sand. Raw sludges perform even less desirably because of grease, oils or other fatty content which clogs the sand. Odours are more prevalent with raw or partly digested sludges.

DESIGN CONSIDERATIONS

Some of the principle considerations which will determine the applicability of drying beds are:

population served
industrial wastes present
location
availability of suitable land
climate
hydraulics

a) Population

As already indicated, sludge drying beds have their greatest application at plants which serve populations ranging up to 25,000 persons. In spite of this, beds are used for both small and large populations. The choice of which dewatering device to use is obviously related to other more significant factors.

b) Effects of Industrial Wastes

The presence of industrial wastes which will alter the characteristics of the sludge to be handled may discourage the use of drying beds. Generally, wastes which interfere with digestion will affect the operation of dewatering and might exhibit the following characteristics:

- high total, volatile or non-volatile solids.
- floating material.
- greasy or high fatty content wastes.
- high volume wastes.
- extremities and fluctuations of pH.
- colour.

Industrial wastes including high suspended solids wastes, phenolic, acid or alkaline wastes or toxic metals would account for these problems.

c) Location

Odours will accompany the use of sludge drying beds. These usually remain localized. At least two hundred feet should separate the sludge beds from dwellings where digested sludge is dewatered. Dewatering on beds of raw sludge or activated sludge should be in areas isolated from habitation.

d) Suitable Land

The availability of low cost land may be a determining factor in the selection of drying beds. Providing ground water contamination is not critical, natural sand without underdrains may be used. On the other hand, if seepage from the beds may reach water bearing materials, or if the area is subject to flooding, a concrete floor should be constructed.

e) Climate

Plenty of sunshine, little rain and a dry atmosphere are optimum for the successful operation of sludge beds. As expected, the southern or western climates are best suited. Adverse weather will limit the use of open beds, and certain plants have been equipped with glass-enclosed sludge beds. These installations are rarely made today as other methods of dewatering are usually more economical.

Proper programing of the digester and the sludge drying beds will permit fuller use of the beds during the dryer months of the year.

f) Hydraulics

Gravity discharge from the digester is preferred from the point of economy. Proximity to the digester or pumping unit will permit savings in piping. Underdrainage systems must be suitably arranged and positioned above flood levels of adjacent receiving waters.

Determinants of area of sludge drying beds

1) nature of sludge to be dewatered
2) climate

Providing sufficient climatic data is available, the more conservative area requirements which are often stated may be relaxed to suit the application. Usual figures of the area required for beds receiving digested sludge from domestic sewage are given here.

Minimum area in sq. feet per capita

Sludge type	Open Bed Area required per capita	Closed Bed
primary sludge	1.5	1.0
primary and humus sludge	1.75	1.25
primary and activated sludge	2.5	1.5
primary and chemical sludge	2.5	1.5

Similar figures are given for glass covered beds to show the effect of climate.

It will be seen that the bed area requirements for plain sedimentation sludges are less than the requirements for other types of sludges.

Designs are based on the yield of wet sludge expected over the year.

Example of Computation

- suspended solids content of raw sewage - 0.2 lb. per cap./day
- suspended solids removed by primaries - 55%
- volatile content of raw sludge solids - 75%
- reduction in volatiles by digestion - 55%
- solids content digested sludge - 6%
- depth of sludge applied to beds - 8 in.
- number of applications per year - 8

per capita production of raw sludge - 365 x 0.2 x 0.55	= 40.0 lb./yr.
volatile content of raw sludge 75% x 40.0	= 30.0 " "
ash in sludge	= 10.0 " "
volatile solids in digested sludge 30(1-0.55)	= 13.2 " "
total solids in digested sludge	= 23.2 " "

Volume of wet sludge (cu^3) $\frac{23.2 \times 100}{6 \times 62.3} = 6.2$

Bed area required $\frac{6.2 \times 12}{8 \times 8} = \quad = 1.16$ ft^2/capita

CONSTRUCTION OF SLUDGE BEDS

a) Filter Media and Underdrains

A level bed of sand from 9 to 18 inches thick overlies graduated layers of supporting gravel in the bottom-most layer of which is set a system of underdrains. Lateral tiles are spaced 8 to 10 feet apart. Specifications for the sand and gravels may be found in any standard reference manual.

Liquor from the underdrains is normally returned to the treatment process.

b) Embankments

Earthen or concrete embankments are used. These are usually 12-14 in. in height.

c) Piping and Distribution of Sludge on Beds

Piping operating by gravity is laid on a grade to provide a velocity of at least 2.5 fps. Arrangements should be provided for draining and flushing the lines following use. The inlet of piping to the beds usually terminates with splash pads to assist distribution of the flow and prevent scouring of sand.

d) Facilities for Sludge Cake Removal

Dried sludge can be removed by a variety of devices. Small plants will employ wheel barrows, while larger plants may employ concrete runways, for trucks or other arrangements.

MANAGEMENT OF DRYING BEDS

a) Preparation for use

Prior to applying a new batch of sludge from the digester, the following steps should be taken:

1) Remove all old dried sludge
2) Never apply sludge to a bed in use
3) Remove weeds
4) Level the sand and scarify the surface with a rake or similar implement, relevel the sand with a slight slope away from the point where the sludge enters. Working of the top sand layers will ensure against compaction of the media and improves its filterability.

b) Withdrawing Sludge and Application to Beds

Operation of the sludge draw-off from the digester should ensure that the line is kept free at times when the line is not in use. This may require flushing of the line to remove plugging which might create internal pressures of explosive gases produced by digestion in the line. Smoking in the vicinity of open sludge valves which discharge to the atmosphere should be prohibited.

Sludge deposited in layers from 8 to 12 inches will normally dry in one to two weeks. Thin layers will dry more quickly, but more labour is necessary in handling a given volume of sludge. The higher solids content sludges will require longer drying periods.

c) Removing of Sludge Cake

Usually, the need to remove the next batch of sludge from the digester and/or the moisture content of the sludge cake will determine when removal is required. Removal at moisture contents of 40 to 70 per cent is usual. Sand will cling to the sludge cake and replacement from time to time is required. At least four inches of sand should be maintained on the beds to provide sufficient filtering action.

d) Records

Records of the sludge drying operations should include the following:

1) total gallons drawn to beds
2) average pH, % solids and % volatile solids
3) cubic yards of sludge removed
4) drying time in days

e) Use of Alum for Conditioning

Normally, chemical conditioning of sludges to be dewatered on sand beds is not required. The dewatering of certain sludges may be improved using alum to assist in the initial **gassing** period. Carbon dioxide is released when alum and the carbonate salts in the sludge react. About 1 lb. of alum applied to 100 to 300 gallons of sludge just before introduction to the beds has proven effective.

BUILDING AND GROUND MAINTENANCE AT PLANTS

by

A. Clark

Operations Engineer - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
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April 23, 1963

BUILDING AND GROUND MAINTENANCE AT PLANTS

by

A. CLARK

Operations Engineer

INTRODUCTION

A prime requisite for any plant, treating or pumping sewage, is that it be clean. I should mention here that there is an immense psychological value in keeping a plant in a high state of cleanliness. All visitors do not expect a high standard of cleanliness in the sewage plant or pumping station and some come to the plant for this sole reason, and if of a critical nature, are usually pleasantly surprised. Round one to the operator - If he applies what he has previously learned in public relations, there is a good chance that he will win round two and any subsequent round. With a dirty plant, you may as well take a drive.

PART A

BUILDING MAINTENANCE

A large part of this is housekeeping. Cleaning should be done on a routine basis. Particular emphasis should be placed on office space, lunch room, washroom and locker room. Close attention should also be paid to laboratory facilities and control area. As well as being kept clean, these areas should be kept tidy.

Next, in the order of importance, are the areas such as blower rooms, pump rooms and chlorinating rooms. Whenever possible, floors in these areas should be kept waxed and polished. A non-skid wax should be used. Keeping floor areas clean in winter time is a problem, especially if the operator has to run out-and-in answering the telephone, fetching tools, etc. but it must be done. The floors can be protected by coconut matting, or paper runners in the main tracking areas. In the modern plant, an industrial type floor polisher is almost a necessity and if you do not have one, you should make every effort to persuade your employer of the need of one.

(A household polisher is no good - too light -- Industrial model if areas large)

Next, in order of importance and forgotten by everyone but the visitor, would be ledges, desk tops and shelving. These should be kept clean at all times. Some visitors like to look in cupboards. Keep in mind that cupboards are for storage and not for depositing odds and ends in a haphazard fashion.

Next on the list would be windows. Windows are for two basic purposes: -

a) to permit light to enter the building, and --

b) to enable one to look out.

When windows become dirty, efficiency in both these items is lost. It should be noted that windows consist of two sides - inside and outside - and both get dirty.

The operator can be aided in his cleanliness campaign by routine painting and waxing. Scaling pipe never looks clean and an unwaxed floor is difficult to maintain. This waxing of floors can be carried one step further. We have carried out some experiments with the waxing of concrete floors in such unlikely places as pumping stations, drywells and basements of control buildings. The basic reasoning is sound. Any concrete floor will continually dust from the day it is poured. Dusting becomes worse as time progresses. The wax seals the concrete from erroding action of the atmosphere. A good concrete sealer would have the same effect but it will not shine up under polishing as is the case from an application of a liquid wax. In one large pumping station, where the operator brushed the floor daily, he found this tedious and applied the recommended right kind of liquid wax and polished it. He now hoses down this same floor twice a week.

Laboratory equipment, especially glassware, should be kept clean when not in use. Special solvents are available for cleaning glassware. If lab equipment only consists of three Imhoff cones, the same attention should be given to the cones.

Equipment painting is a subject by itself, and this lecture will confine itself to the appearance rather than the protection aspect. The basic rules apply: -

1. Do not paint over rust.

2. Do not paint over partially scaled paint, and if paint continually peels, find out why.

3. Paint work should almost always be cleaned with a stiff wire brush or sand blasted, wiped, an application of primer, followed by an application of finish coat.

There is a fairly general opinion that copper and brass fittings and piping should be burnished and not painted. If they are burnished, a light coating of clear varnish will maintain the appearance indefinitely.

Painting of concrete floors can be a risky business. What will give satisfaction in one plant gives heartache in another. There is one plant in Ontario where all forms of pre-treatment have been tried and the most expensive paints used, and within three months, the paint has scaled-off. However, concrete which has been well cured and received a wearing surface receives paint readily and general appearance is satisfactory.

PART B

OUTSIDE MAINTENANCE

Still confining our discussion to buildings, we will examine what can be done, and any little tricks we may apply to aid the appearance with a minimum of additional effort.

When we discussed windows, we got both sides clean, so we will leave them. The brick work, when it becomes splattered with mud, foam residue or plain rain soil, should be hosed and occasionally brushed. It may be necessary, on occasion, to wash the brick work down with muriatic acid especially if efflorescence is the problem. Where vertical siding is used as an architectural feature, it is much easier kept oiled than varnished. If you are not already keeping your tanks hosed and brushed, it is too late to mention them, however, here are a few thoughts on effluent launders.

Effluent launders can be painted either with a metallic or rubber base paint. Good use can be made of ceramic tile especially in white or pale colours. This is not as ridiculous as it sounds and it takes you one step closer to that mythical clear sparkling effluent. Whatever your choice of paint or tile, they are much easier kept clean and in many cases a rubber squeegee can be used in preference to a wire brush. I would suggest to you that wire brushing of launders produces algae growing conditions. With a smooth sealed surface, algae has difficulty finding a resting place.

Collector mechanisms and clarifiers should be hosed daily and so should scum troughs. In the case of mechanical aeration; draft tubes and mechanical drive units should be cleaned regularly. With the present foaming conditions at plants,

effluent water spray systems should be installed to keep down the foam. By using the effluent water, we may avoid unnecessary water bills. However, it may be adviseable, on individual occasions, to hook up a spray system directly with the municipal water supply. Uncontrolled foaming will spoil building appearance, discolour concrete and kill most plant life.

Handrails around tanks should be kept clean and painted and lucky is the operator who inherited aluminium handrails.

All this endeavour is time consuming, but the Chief Operator who has had his plant spotless for an official opening will confirm the old adage that it is easier to keep a clean plant clean than to keep a half clean plant half clean.

GROUND MAINTENANCE

(A) Grass Cutting - Area is the criterion. Small areas can receive complete attention, large areas should receive only graded attention. If we consider a small area of one acre or less - this presents no problem since an area of this size can be kept close trimmed and weeded at all times. However, when the area is greater than this, in many cases up to ten or more acres, the operator does not have the time nor staff to keep the entire area in bowling lawn condition. Where this is the case, concentration of attention should be on the area adjacent to buildings, tanks and access roads. This area should extend some 25 to 50 feet beyond buildings and tanks. In the case of access roads, anything up to 25 feet on either side of the access roads including the ditches produces the desired appearance. This area should be kept close trimmed and weed free at all times. Grass beyond this area should be kept reasonably short and cut down when it grows to or reaches a height of approximately 6 inches long and raked. There should be no concentration of watering or weeding, but it might be possible to extend the cutting to once every two weeks.

In most plants this would extend to the fence line or even outside. At many plants, property beyond the fence is owned and if this is a field, consideration should be given to renting out or lending it to a farmer. Consideration to the growing of hay might even be given. In some localities hay is selling; in the fall or mid-winter season for 50¢ a bale and about 25 bales can be harvested from an acre. Hay in itself is not an unsightly thing.

(B) Landscaping - The immediate building area extending 25 to 50 feet beyond buildings and tanks should be landscaped. As previously mentioned, grass should be close cut and weeded. I would caution you here that close cutting -- it is better to

keep the grass no shorter than one inch for in many cases this leads to strengthening of the grass and control of weeds by natural processes. The use of gravel or crushed stone adjacent to the tanks is serviceable and if foaming is a problem, there is no plant life to be killed. However, foam landing on gravel areas soon makes a black slimy mess that is difficult to wash off - so the idea is to keep foam down. The growth of weeds may be a problem in a gravelled area. This can be overcome by laying plastic under the stone.

Lawn areas are the most attractive and catch the eye readily and should be kept clear of weeds as previously mentioned. There are numerous selective weed killers on the market and the operator has a wide choice of what he wants to kill. (His ultimate objective might be to kill the grass.)

On watering -- it can become quite expensive if the operator is unfortunate enough to be on the metered system and if there is a water shortage in the area it would not look too wise for the sewage plant to be watering lawns when everyone else is being told to economize. However, most plants have unlimited water at their disposal --namely the effluent -- and in many cases, the receiving stream would show its gratitude by improving in appearance - if effluent water is used on lawns. Consideration should be given to the use of a high pressure effluent water pump, such as is used in the spray systems or it may be possible on occasion to use some of the foam control spray water for lawn watering. Most authorities recommend that watering be done during midday (when foaming at the plant is not a problem). The reason they give for lawn watering is that grass watered in the evening will sour overnight.

Access roads, pathways and driveways are frequently neglected, often through no fault of the operating staff. It is impractical to have a gravel access road on which liquid sludge trucks make daily trips. After several years of this operation, the access road assumes the appearance of a cowpath and very little can be done with it except regrade it. Under these circumstances, lightly travelled roads should be oiled and heavily travelled roads should be paved.

Flower beds enhance the plant appearance but are time consuming and <u>should be kept to a minimum</u> and restricted to locations where they immediately catch the eye. In planning flower beds, do not restrict the flowers to one species. Rather, arrange them so that there is a group of flowers in bloom through the summer season. <u>Perennials</u> require less attention and less financial outlay than the annuals.

Trees and shrubbery should receive close consideration, especially beyond the immediate landscaped area. Trees have two basic functions -- wind breaking and screening. As you all know,

there are two basic types of trees in this area: -

a) coniferous

b) broad-leafed

(A) <u>Coniferous</u> trees would be -

1 - Eastern White Pine - which is a tall, stately tree and can reach a height of 175 feet with a trunk diameter of 5 feet, but this is unusual and it is usually 100 feet or less in height and no more than 3 feet in diameter. This tree will grow on wide range of sites from dry sandy to bogs, but it does best on a moist sandy soil.

2 - Red Pine - Its average height is 100 feet with a trunk diameter less than 3 feet. It grows best on a loamy sand or gravel but can thrive just about anywhere.

3 - Tamarack or Juniper - This is a medium size tree 60 to 70 feet high with a **trunk** diameter up to 2 feet. This tree grows best in damp, swampy areas.

4 - White Spruce - This can grow to a height of 120 feet with a trunk diameter of 4 feet (usual height is 80 feet). This tree grows best in well drained moist soil close to streams.

5 - Hemlock - This is a medium size tree growing to some 60 to 70 feet in height and up to 2 feet trunk diameter. This will grow just about anywhere.

6 - Balsam Fir - This also is a medium size tree and it will grow just about anywhere.

(B) Broad-Leaf Trees

1 - Aspen or Poplar - The average height of this tree is about 40 feet with a trunk diameter of about 8 to 10 inches. This tree grows best on a well drained loam and is found almost anywhere.

There are other broad-leafed trees, some of which are:- Walnut, Hickory, Iron Wood, Birch, Beech, Oak, Elm and Maple.

I have mentioned only a few of the possibilities. There is an infinite variety of trees, but the ones that I have mentioned are readily adaptable to Southern Ontario climates and the Evergreens, of course, will grow to the far North of Ontario. Operators with plants on the North shore of Lake Superior should take the advice of the local Forest Ranger. Broad-leafed trees should ~~not~~ be planted adjacent to tanks or they create a leaf problem in fall with subsequent plugging of pipes and pumps.

Those operators with small plants will not be able to go into the tree growing aspect and should consider instead, the growth of shrubs. Here again, climatic conditions prevail, but there are numerous Evergreen shrubs that can be grown easily in Southern Ontario.

LAWNS

(A) I MATERIALS AND TOOLS

A) Equipment for Buildings or Renovating a Lawn:-

1 - Humus and fertilizer - Any texture of soil will suffice in a pH of between 6 and 7 if necessary. Starting with 50 lbs. of fertilizer per 1000 sq. ft. - first year - it should be fed at the rate of 20 lbs. per sq. ft. per year, preferably in March, April and September.

2 - Grass seed of good grade in a mixture suitable to soil and light conditions - Quantity -- about 4 lbs. per 1000 sq. ft. Tools and machinery -- a Roto-tiller; spade, if the work is to be done by hand; iron rake; baskets, wheel barrow, roller or tamper; and a hose.

II PROCEDURE

A) Building a New Lawn: -

1 - Choose the most opportune time to sow seed. Early fall, late August or September is best, with early spring as a second choice. Rye grass or Timothy can be sown temporarily - to be ploughed or dug under later, or sodding may be resorted to - although it costs twice as much as seeding.

2 - Prepare the seed beds some weeks in advance so that it can settle before sowing.

General Procedure:-

a) - Test soil to find out if lime is needed and how much. (Check the pH)

b) - Plough or Roto-till the existing soil.

Remove stones and debris and rough grade before preparing the seed bed.

c) - Replace any top soil which may be piled to one side of the lot during rough grading for drainage, etc.

d) - Spread (all at one time, if desired) any necessary lime, commercial fertilizer humus (sludge) and mix them with the top 6 inches of soil by Roto-tilling or digging it at least a week in advance of seed sowing.

3 - Other good grade of lawn seeding have a mixture which suits soil and light conditions. If the grounds are in part shady areas, then a shady mixture may be sown throughout. If both sunny and shady mixtures are being used, grade the two together where they meet to avoid the deciding difference in grass texture.

4 - Add more fertilizer to the surface inch or two, rake it in and smooth down the surface just prior to seed sowing.

5 - Sow the seed in two directions, first splitting the quantity equally.

6 - Rake the seed into the soil very lightly after sowing and firm the bed with a light roller, if available, or a tamper.

7 - Water with a spray mist if drought sets in before the seed sprouts well. Supply ample water throughout the first season.

8 - Begin to mow when the grass reaches a height of 3 or 4 inches. Set the mower to mow the new grass at $1\frac{1}{2}$ inches height.

9 - Destroy crab grass and other weeds when they appear. Do this by hand the first season as chemicals would be too strong for new grass.

10 - Follow through every year by feeding and top dressing in the spring or fall and by keeping the grass as weed free as possible.

B) Renovating an Existing Lawn:-

If a lawn has a stand of 1/3 or more of good grass, it is worth renovating. The following method can be used to improve a new lawn if poor thin soil, or an old lawn which has become crab grass infested.

Early fall is the preferred time to renovate, otherwise early spring.

1. - Mow the grass short first then rake off the clippings along with the fallen leaves, dead crab grass and other weeds.

2. - Feed the whole lawn with a special lawn fertilizer or general commercial fertilizer (sludge, supernatant, or waste activated sludge).

3. - Scratch all the bare patches of soil with an iron rake.

4. - Seed them with a good mixture by hand and rake in very lightly.

5. - Using a light roller or tamper, compact the soil.

6. - Keep the surface soil moist by watering, both before and after the grass shows above ground. (Plant Effluent).

SAFETY IN THE HANDLING OF CHLORINE

by

W. P. Carter

Technical Service Supervisor
Chemical Division - Sales
Canadian Industries Limited

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 23, 1963

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Chlorine is one of the most important chemicals of our modern industrial world. It was discovered in 1774 by C. W. Scheele, a Swedish chemist, who regarded it as a compound substance containing oxygen. In 1810 it was identified as an element by Sir Humphrey Davie who gave it the name of Chlorine, derived from Chloros, the Greek word meaning green. Commercial quantities of Liquid Chlorine were not produced and shipped until 1888 when it was found that the dry Chlorine gas did not attack iron and steel. The Canadian demand for Liquid Chlorine has increased from approximately 10 tons per day in 1922 to 850 tons per day in 1959.

Chlorine is prepared from a salt brine by an electrolytic process in which the brine decomposes to form Chlorine, Caustic Soda and Hydrogen. Chlorine is liberated at the amode, a positive electrode of the electrolytic cell, and drawn off through stoneware pipes to the drying and liquifying plant. All traces of moisture are removed from the Chlorine and then by a process combining pressure and refrigeration the gaseous Chlorine is loaded into cylinders, ton containers and tank cars for shipment to consumers. Under normal atmospheric conditions Chlorine is a greenish yellow gas with a penetrating and characteristic odor two and one-half times as heavy as air. Chlorine by itself is non-flammable and non-explosive. While Chlorine is not combustible in air it will support the combustion of certain compounds.

Chlorine unites directly with most of the elements to form chlorides. The reaction is rapid if the Chlorine is hot or moist thus, to avoid corrosion, Chlorine is handled in a cool dry state. Corrosive attack is due to the action of Hydrochloric and Hypochlorus acids formed by hydrolysis. Chlorine reacts in solutions of alkalies to form hypochlorites, such as Calcium Hypochlorite prepared from lime and Chlorine, and Solium Hypochlorite prepared from an aqueous solution of Caustic Soda and Chlorine. Chlorine by itself is not explosive nor is it flammable but it will support combustion.

For example, steel at a temperature of 483°C. will ignite in the presence of Chlorine and burn quite readily. Other metals will act similarly at slightly higher temperatures.

To meet customers' requirements, Chlorine is generally shipped in the following sizes of containers:

<u>Cylinders</u> - containing 150 pounds.

<u>Ton containers</u> - capacity - one ton of Liquid Chlorine

<u>Single unit tank cars</u> - capacities of 16, 30 and 55 tons.

All Liquid Chlorine containers are constructed to conform to the specification of the Board of Transport Commissioners for Canada. They are tested at regular intervals according to the requirements prescribed by the Bureau of Explosives. In this talk we will restrict ourselves to cylinders and ton containers since these are the containers in which most of you are interested.

Chlorine cylinders are of seamless steel construction and each is equipped with an approved Chlorine Institute valve. In addition, each cylinder is fitted with a bonnet designed to protect the valve from impact due to knocking or dropping. Letters and numbers are stamped on each cylinder indicating ownership, specification, cylinder number, tare weight and dates of hydrostatic tests. It is illegal to deface these markings in any way.

The contents of any Liquid Chlorine container consist of a liquid phase and a gaseous phase. For reasons of safety, the Board of Transport Commissioners has limited the weight of Chlorine which can be shipped in a container to not more than 1.25 times the weight of water which it will fill to capacity. When so loaded at a temperature of 68°F., Liquid Chlorine will occupy about 88% of the container volume and gaseous Chlorine will occupy the remainder. Cylinders are filled and discharged through a valve on the top. This valve has a special bronze body and a monel metal stem. The valve packing is designed especially for the purpose. The body of the valve is fitted with a plug of fusible metal designed to melt at 160°F., thereby relieving the pressure and preventing rupture of the cylinder in case of fire or over-heating. This plug must not be tampered with as the full pressure of the gas is behind it regardless of whether the valve is closed or open.

When cylinders and ton containers are returned to the Works, they are evacuated, thoroughly cleaned, dried and inspected before being refilled for shipment. After cleaning, each cylinder is fitted with a new or thoroughly reconditioned valve.

As an added precaution, every cylinder is retained at the Works at least 24 hours after filling so that any possible defect may be detected prior to shipping.

STORAGE AND HANDLING

Chlorine cylinders should be stored upright and arranged so that any cylinder can be removed with a minimum handling of the other cylinders.

It should be remembered that when a cylinder is in an upright position it will provide gas whereas in a horizontal position Liquid Chlorine is obtained. Cylinders should not be stored under any of the following conditions.

1. Near combustible or flammable materials such as oil, gasoline and waste.

2. On an uneven floor or one covered with debris.

3. Near the inlet of a ventilating or air conditioning unit.

4. In sub-surface locations.

5. Adjacent to any source of direct heat such as furnace heating, element or radiator.

Cylinders should never be dropped or allowed to strike each other. When moving cylinders it is recommended that a light two-wheeled hand truck be used. This hand-truck should have a clamping arrangement or safety chain at least two-thirds of the way up the cylinder.

With experienced personnel a cylinder can be safely moved by rolling it on its bottom edge. When rolling a cylinder in this manner, care should be exercised that it does not get out of control and fall, also that the proctective bonnet does not screw itself loose.

When handling Chlorine cylinders, the protective bonnet should be in position and should not be moved until the cylinder is to be connected to the chlorinating system. The same care should be used in handling full and empty cylinders.

When it is necessary to lift a cylinder and an elevator is not available, a crane or hoist with a special cradle should be used. Chains, rope slings and magnetic devices should never be used. Never lift or support a cylinder by means of the protective bonnet because the hood is not designed to carry the weight of the cylinder.

Care must be exercised in unloading Chlorine cylinders from truck transports where there is no receiving dock. Under no circumstances must the cylinders by allowed to slide off the end of the truck to ground level as rupture of the cylinder could occur. The cylinder should be unloaded by means of a hydraulic tail gate or, if this is not available, by a skid.

CONNECTING CYLINDERS

Chlorinating equipment is frequently supplied with a union connection. When this union is used to connect directly with the cylinder both union and valve threads will in time become worn or damaged so that a tight connection is impossible. The use of clamps and adaptors with lead washers for connecting to cylinders eliminates this trouble. This equipment is provided free of charge by the Chlorine producer. When connecting the cylinder to the chlorinating system, it is recommended that the steps listed below be followed:

1. Secure the cylinder to a building column or a solid upright support.

2. Remove the protective bonnet. If cylinder has been exposed to the weather for a long time, the threads at the base of the bonnet may have become corroded, in which case, a few sharp raps on the opposite side of the bonnet will loosen it so that it may be unscrewed easily.

3. Remove the brass outlet cap and any foreign matter which may be in the valve outlet recess. When removing the brass outlet cap, it is a good idea to do it carefully and have on eye protection, as there is a remote possibility of Chlorine being trapped under the cap.

4. Ensure that a new lead washer is in the outlet recess.

5. Place a clamp over the valve and insert the adaptor in the outlet recess, and then fitting the adaptor in the clamp slot, tighten the clamp screw.

Make sure that the end of the adaptor seats firmly against the lead washer. These valves are designed primarily as shut-off valves, not control valves, so there should be a control valve somewhere else in the system. The correct procedure for opening the cylinder is to place the special wrench on the valve stem, stand behind the valve outlet, and while grasping the valve firmly with the left hand, hit the wrench a sharp blow in a counter-clockwise direction with the palm of the right hand. Do not pull or tug at the wrench, as this may bend the valve stem causing it to stick. Under no

circumstances should an extension be added to these wrenches as with the extra leverage obtained there is a danger of bending the valve stem, or even sheering it off.

Before opening the valve it is desirable to see that the packing nut is snugly tight, and after the valve is opened and the flow of Chlorine adjusted, this nut should again be adjusted and at the same time a check made for any leaks with an Ammonia bottle. The maximum discharge rate for a single 150# cylinder is about two to three pounds per hour. A cylinder when discharging gaseous Chlorine is actually refrigerating itself. As the Liquid Chlorine inside the cylinder vaporizes, it takes up heat from the surroundings. You will have noted that when a cylinder is nearly empty, frost forms on the outside. This acts as an insulation, and reduces the amount of heat the cylinder can get from its surroundings. This condition lowers the Chlorine pressure considerably. This frost may be removed by scraping or by circulating warm air around the cylinder with a small fan. Do not under any circumstances, apply hot water or external heat to a cylinder to speed up the supply of Chlorine. This is a very dangerous practice. The recommended method of improving slow delivery of Chlorine is to connect one or more cylinders to a manifold.

In this connection, we might mention that when discharging cylinders through a common header, care must be taken that all cylinders are of the same temperature. This applies especially when connecting a new cylinder to a header. If there is a difference in temperature of the Liquid Chlorine in the connected cylinders, Chlorine will transfer by distillation from the warm to the cool cylinder and may result in its becoming completely filled with Liquid Chlorine. This would be extremely dangerous as the cylinder might rupture if the valve on the over-filled cylinder were closed and the Chlorine temperature then raised.

The only reliable method of determining the contents of the cylinder is by weighing. The pressure in the cylinder depends upon the temperature and not on the amount of Chlorine in the container. Where convenient, it is recommended that the cylinder stand on a scale throughout the entire period of discharge. In any event, the only sure method of determining whether or not the cylinder is empty is to weigh the container and check its weight with the tare stamped on the cylinder shoulder.

The procedure for closing the cylinder valve is essentially the same as that described for opening it. If the valve does not close tightly on the first trial it should be opened and closed tightly several times until the proper seating is obtained. Under no circumstances should a hammer or other implement be used to effect a tight closure. The outlet cap should be replaced and the protective bonnet screwed into place.

The final step in returning an empty Chlorine cylinder requires that the portion of the green warning tag below the perforated line be removed. By this means the cylinder is identified as an empty container. If a full or partially full cylinder is being returned, then the green warning tag should be left intact.

Ton containers, like 150# cylinders, must be handled and stored in a safe manner. These containers are stored in a horizontal position and in a dry well-ventilated location. The use of a trolly and hoist of at least two tons capacity on an I. beam mono rail is recommended as the best method of handling these containers. A special grab, fitted with hooks which fit into the chimes of the container, is used for lifting.

Ton containers can be shipped by (1) transport, in which case they must be securely chocked. (2) by multi-unit tank car which consists of 15 x 1 ton containers on a specially designed railway flat car and (3) since the first of this year, on a specially designed truck with a self-unloading mechanism which permits tonners to be lowered to the ground and empties picked up by the same device. The use of this equipment will in some cases mean that the expensive mono rail arrangement is not required.

A ton container has two valves very similar to those on the 150# cylinders, in fact the only difference is that these valves do not have fusible plugs. The ton container has three separate fusible plugs in each end. The metal plug in these will melt at 160°F. The positioning of the valves on a ton container is important, when the two valves are in a vertical position, the top valve delivers gaseous Chlorine while the lower valve delivers Liquid Chlorine. The delivery rate of chlorine from a ton containers, of course, depends on the temperature, but an average flow is about 20# per hour of Chlorine gas at room temperature.

MATERIALS OF CONSTRUCTION

When shipped, Chlorine is substantially free from moisture. In this dry state and at ordinary temperature, it does not attack most common metals, zinc and tin being notable exceptions. Most metals, however, will corrode at a rapid rate when exposed to wet Chlorine.

Chlorine combines with water to form Hypochlorous acid, a powerful oxidizing agent, which few metals can withstand. Leaks occuring in the presence of water or moist air tend to increase rapidly in volume due to the corrosive action of this acid.

Pipe lines for handling dry Chlorine as a liquid or gas should be fabricated from Schedule 80 seamless steel pipe although copper piping may be used if desired. Gaskets made of asbestos or lead are recommended. It is recommended that lines be of welded construction and ammonia flanges installed at intervals to facilitate dismantling the system for inspection or repair. The use of rubber gaskets in Chlorine lines should be avoided. To provide for a flexible connection between the Chlorine container and the pipe line, and between the pipe line and the chlorinator, an upright coil of copper tubing is recommended. Copper tubing, however, becomes hard and rigid after continued use and should be replaced at regular intervals.

Where screwed fittings or joints are required, it is essential that all threads be clean and well formed. Graphite, red lead or Litharge may be used as pipe dope. In making joints, care should be taken to ensure that very little pipe dope is allowed inside the pipe as the oils will be chlorinated to form a gummy substance which may clog in the system. Similarly, all cutting oils should be removed from the inside of the pipe.

In the case of larger consumers of Chlorine it may be necessary to withdraw liquid from the containers and vaporize it in a vaporizer. In this case, precaution should be taken to ensure that at no time is Liquid Chlorine in a line shut off at both ends with a valve, without having protected this line with an expansion chamber, thereby preventing hydrostatic pressure from rupturing the line. As a general safe guard, Chlorine lines should be subjected to a 300 pound hydrostatic pressure test before the line is put into service.

If water has been accidentally introduced into a Chlorine line or if it has been opened for repairs or cleaning, corrosion can be kept to a minimum by immediate drying. It is recommended that the following procedure be observed:

1. Pass steam through the line from the high end, allowing condensate, corrosion products and chlorinated impurities to drain out. This should be continued until the line is thoroughly cleaned.

2. Disconnect air supply.

3. Blow immediately with dry air, while the line is still hot. To ensure that all moisture is removed, it may be necessary to continue blowing the line for several hours.

Underground lines should in general not be used for Chlorine. Such lines introduce additional hazards and complications which are expensive to overcome. If however, underground lines are unavoidable, the Chlorine suppliers will be pleased to supply detailed information on the proper method of installation.

Valves in Chlorine service can be globe, angle valves or in the case of small lines, bar stock needle valves. These valves should be of bolted bonnet construction with outside screw and yoke. The body, bonnet and yoke should be of forged carbon steel with monel or Hastallory C trim. The packing for Chlorine valves is normally graphited asbestos although Teflon is rapidly becoming popular.

A recent survey among the Chlorine producers in North America showed that between one and two cylinders in every thousand develop minor leaks after leaving the supplier's plant. This, despite the extreme care which is taken in the cleaning, inspection, filling and valve maintenance and overhaul on all cylinders. I hope to be able to show you by means of slides, if time permits, the procedures followed at our plant at Cornwall, Ontario. Most of the troubles arise from two causes, i.e. a turning spindle or leaking fusible plug. A turning spindle is actually a stripped thread on the valve body and results from the fact that the valve stem is made of monel metal whereas the valve body is made of softer bronze. This defect is normally discovered when trying to close off a cylinder already in service and the simplest way of overcoming this problem is to continue using the Chlorine until the cylinder is empty. However, under some circumstances this is not possible and one way of handling the situation is to disconnect the cylinder in such a manner as to include a second valve with the unit disconnected. This valve is then acting as a shut-off valve. The cylinder may then be moved to a less hazardous location; at the same time a call should be put through to the Chlorine supplier who can then bring special equipment to deal with the cylinder. It is also possible to gradually bleed the cylinder to atmosphere or to bleed the Chlorine into an alkaline solution using a rubber hose or steel pipe. For this purpose, Caustic Soda, Soda Ash, lime or other alkali can be used. These solutions will absorb Chlorine at a fairly rapid rate. The other type of a leak which occasionally occurs, is when a fusible plug develops a leak. This is usually due to corrosion either internally or from outside. Once again, the supplier's personnel can be called in and apply a suitable clamp over the fusible plug to stop the leak. However, it is also possible to use the standard adaptor clamp for this purpose. Procedure is to use a flat file on the area around the fusible plug flat, apply a small patch of rubber gasket material with a follow-up piece of steel and clamp this firmly in position. It should be noted carefully, however, that the cylinder is now without protection from high internal pressure so that every attempt to use this cylinder up as rapidly as possible should be made.

A third possibility of cylinder leak could occur around the valve packing but in this case, with the valve shut off, the packing nut may be removed and the valve repacked without any danger. Sometimes merely tightening the packing gland nut will suffice.

As mentioned previously, it is very important that a leaking Chlorine cylinder be positioned so that gas rather than liquid is escaping. In other words, the cylinder should at all times be kept in a vertical position.

In addition to the above more or less normal type of leak, I would like to mention the following somewhat special situations which can represent serious potential hazards. A leaking cylinder should never be lowered or dropped into a shallow tank or small stream or pond with the idea of absorbing the Chlorine in the water. Actually the solubility of Chlorine in water is very slight, and in the case of a small leak through the fusible plug, the opening is quickly enlarged by external corrosion. Then since the cylinder is in a horizontal position, Liquid Chlorine is forced out in such large quantity that the water is able to absorb only a small percentage of the gas evolved and a very serious situation results.

Another serious potential hazard can develop if a cylinder is connected directly to a high pressure water main, as in the case of chlorinating new water distribution systems, without the use of a chlorinator. If the water pressure in the main is higher than the Chlorine pressure in the cylinder, water will be forced back into the cylinder. When this occurs heat is generated, often sufficient to melt the fusible plug, thereby releasing Chlorine gas. At the same time rapid corrosion results and instances have occurred where the metal wall of the cylinder has been completely eaten away.

Direct chlorination of water mains is sometimes attempted by a contractor often without a chlorinator. This work should be done only by competent water works personnel who by proper operation of valves can control pressure in the section of line being chlorinated, which should never exceed the pressure in the cylinder.

Chlorine lines can be broken through accidental mishandling or chlorine can be discharged into the chlorinating room by leaving the wrong valve open, etc. When working with Chlorine care should be taken to follow the prescribed procedures so that mistakes are not made. The Chlorinating room itself should be adequately ventilated with a separate ventilating system usually able to remove the air from the room once every four minutes. The outlet should be located at low level in the room because Chlorine gas is heavier than air. Ideally, there should be two exits from a room containing Chlorine containers, and the doors should open outwards to facilitate rapid exit. Suitable gas masks should be available in case of emergency. Gas masks should be located just outside the chlorinating room as should be the switches for the ventilating system. Respirators of the cannister type do not supply oxygen, but merely absorb the Chlorine present in the air breathed. This type of mask will provide adequate protection in Chlorine concentrations not

exceeding one percent by volume. If heavier concentrations are encountered, the use of a self-contained breathing apparatus is essential.

As most of you are no doubt aware, a little Chlorine goes a long way. For example, the least detectable odor is about $3\frac{1}{2}$ ppm. The maximum amount that can be inhaled in one hour without serious effect is about 4 ppm. 15 ppm would cause irritation of the throat, 30 ppm will cause serious coughing spells. 40 to 60 ppm is considered extremely dangerous for one-half hour exposure. If you were to take a few breaths of air containing 1000 ppm it is very likely they would be your last.

FIRST AID

Prompt treatment of person exposed to Chlorine is of the most importance.

1. Obtain medical assistance as soon as possible.

2. Inhalation - Anyone overcome by or seriously exposed to Chlorine gas should be moved at once to an uncontaminated area. The patient should be placed on his back with head and back elevated. He should be kept warm using blankets if necessary. Rest is essential.

ARTIFICIAL RESPIRATION

If breathing appears to have ceased, artificial respiration should be started immediately using the Neilson arm lift back pressure method or the Shaeffer prone pressure method.

Milk may be given in mild cases as a relief from throat irritation. Never give anything by mouth to an unconscious patient.

CONTACT WITH THE EYES

If even minute quantities of Liquid Chlorine enter the eyes or if eyes have been exposed to strong concentration of Chlorine gas, they should be flushed immediately with copious quantities of running water for at least 15 minutes. Never attempt to neutralize with chemicals. The eyelids should be held apart during this period to ensure contact with water with all accessible tissues. If a physician is not immediately available, continue eye irrigations for a second period of 15 minutes. After the first period of irrigation is complete it is permissible, as a first aid measure, to instill into the

eyes two or three drops of a .5% solution of Pontocaine or other effective aqueous topical anaesthetic.

The above recommendations are general only. Greater detail will be found in various publications issued by the suppliers and the Chlorine Institute.

MECHANICAL DIFFICULTIES OF DIGESTER OPERATION

by

P. J. Osmond

Project Engineer - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 24, 1963

MECHANICAL DIFFICULTIES OF DIGESTER OPERATION

by

P. J. OSMOND

Project Engineer

In a lecture of this type, it is best to go over the existing types and items of equipment which are associated with digester operation one at a time. Therefore, in order to do this, I would like to follow the sludge through its digestion process from its beginning in the primary clarifiers to its end as a fully digested material immediately previous to disposal, picking up the various pieces of equipment associated with this process along the way.

Primary Mechanism

Now, we see that the first piece of equipment which is involved in this sludge digestion process is the primary clarifier itself, particularly the scum skimming and sludge scraping mechanisms. These mechanisms are primarily designed to remove scum and sludge from the primary clarifier. However, a very important secondary consideration is that these devices should also help to concentrate this material--the scum and sludge--to as high a solids content as possible in order that valuable digester capacity will not be needlessly wasted. A scum skimmer riding too low, or a swing pipe dipped too low in the sewage will tend to convey more liquid than is necessary to transport the scum from the tank to the scum pit, thus resulting in a low overall solids content. On the other hand, a scum skimmer riding too high may not convey enough water for the transfer of this material to the scum pit. Sludge scrapers, in order to concentrate the heavy solids at the bottom of the primary tank, must be properly balanced and positioned so that hit-and-miss action does not occur. A poorly adjusted scraper mechanism will most probably result in sludge concentration variations around or along the primary tanks. This can then cause sludge of a low solids content to be directed to the raw sludge sump in the primary tank. It is, therefore, important that skimming and scraping operations be well adjusted.

Raw Sludge Pumps

We will now move on to the raw sludge pumps which are almost exclusively of the positive displacement type. By far the most common type of positive displacement pump is the piston type pump which I like to compare to the "Turtle" in that old fairy tale entitled the "Turtle and the Hare" -- slow, sluggish but awfully dependable. These pumps, however, are not infallible. Perhaps the most common difficulty experienced with these pumps is blockage of the ball check valves which are ~~associated~~ with and not an integral part of the pump itself. Small stones, twigs, grease, sticks will get under and around these balls and prevent them from sealing. The same is true of the surge tanks which are associated with this type of pump. Stones, hair mats, etc. can block the inlet to these tanks and prevent them from operating. Grit can also wear-out a ball prematurely. As time progresses with this problem wear will take place at an ever increasing rate. It is therefore, necessary that periodic checks be made on the ball check valves and surge tanks in an effort to prevent situations like this from occurring.

Another problem associated with the piston type pump is a lack of lubrication between the piston and the cylinder wall. Because of the nature of raw sludge, scoring of the piston as well as the cylinder wall is very likely, if proper lubrication is not provided. This scoring can reach such an extent that excessive leakage will occur around the piston, thus causing an inefficiently operating pump - not to mention the associated mess, odours, etc. which would eminate from this fault.

Another piece of equipment which is sometimes found downstream of the raw sludge pumps is the elbow type sight glass. This reminds me of a very amusing incident in which I was involved at one of the OWRC operated plants in the summer of 1961. --- Our regular operator was on holidays and in order to take up the slack, we had sent one of our summer students from head office to the plant to help fill in. This fellow, at the time that I was at the plant, was pumping raw sludge and he commented to me after looking at this sight glass that it didn't seem as if there was any sludge going through, I said, "Well maybe you have a valve closed." So this fellow went over and looked at the discharge valve on the sludge pump and said, "My god it <u>is</u> closed", and he immediately swung the valve over to what he thought was the open position. The valve was open in the first place and I don't think that I need to tell you what happened, we were cleaning sludge off the ceiling for about half an hour after that.

Sludge Re-circulation Pumps

Raw sludge is either pumped directly to the primary digester or indirectly through the heat exchanger. Now, since Ray Norton will be dealing with the heat exchangers later on today, I don't think that I will mention this, and we will assume that we pump raw sludge directly into the digester. Therefore, the next piece of equipment that fits into this flow pattern is the sludge re-circulation pump. This pump is normally coupled to the heat exchanger but can be operated independently and is most commonly of the centrifugal type, but can be of the piston type. Since piston pumps have been discussed previously, we will then deal with the centrifugal type pump. The most common problem I have run across with these pumps is scoring of the pump shaft in the area of the packing, thus causing excessive leakage up through the packing past the inboard bearing and usually out onto the floor. Now, since these pumps are normally continuously operating, either to provide circulation for the digester or to maintain the heat in the digester, it is essential that the packing for this pump is well lubricated. Now there are two lubricants that can be used for this job, grease or water -- grease can either be applied directly with a grease gun at periodic intervals; through a spring loaded grease fitting or through a discharge pressure loaded grease fitting. My experience has been that the most efficient method of lubricating the packing to prevent this scoring is hand greasing with a grease gun at periodic intervals. Spring loaded fittings are always losing pressure and discharge pressure loaded fittings tend to clog up. They cannot be left unattended for any extended time, so my view on this is that if they cannot be left unattended, take them off, and you might as well be greasing with a grease gun.

Perhaps the most efficient way of lubricating this pump is through a pressurized water seal system. However, in the event of a failure of this water seal, there is only one alternative that can be taken and that is to connect the potable water system directly to the pump through a garden hose or something similar. However, we all know that this is definitely contrary to the rules and regulations set down by the OWRC governing cross-connections. This is the main reason why I favour just straight grease fittings on these pumps with greasing done manually at periodic intervals.

Another problem arising with these pumps which is more commonly found in the older plant is wearing of the impeller which results in lower volumes of sludge being re-circulated. This can be one cause for inability to maintain digester temperature. I personally have never run into this problem but you never know, perhaps as the plants age this problem may arise.

Valves

In my opinion, one of the most commonly neglected and yet more important pieces of equipment we have in sewage plants are the lowly valves. These valves can be of many various types. Most common in the sludge process is the gate and plug valves. Where a relatively coarse material such as sludge is lodged in a valve, it is necessary that the bearing surface is well lubricated, or else scoring will occur, eventually leading to deterioration of the valves. It is well to keep in mind that the valves should be treated just like pumps. They should be subjected to a normal maintenance schedule; be kept well greased, if possible; and generally kept in good shape.

Other Pumps

Another piece of equipment associated with digester operation is the sludge transfer, sludge loading or digested sludge pump or any combination of the above. These pumps can either be of the centrifugal type or of the positive displacement piston type and what has previously been mentioned concerning these two types of pumps will apply here as well, only possibly more so, since these pumps are handling usually a more dense material. Although this material is more homogeneous than raw sludge, it may have a heavy grit loading depending upon the location of the suction, type of material being pumped to the digester, mixing, etc.

Gas Equipment

The remaining types of equipment which have not yet been discussed and which are still physically exterior to the digester itself are the devices for collecting, controlling and measuring of the methane gas produced in the digester. This equipment includes blow-off valves, vacuum release valves, flame traps, drip traps, gas meters, regulators, waste gas burners, etc.

Now, we all know the purpose of the blow-off and vacuum release valves located on the roof of the digester and I think we all are familiar with the most common problem associated with this arrangement is freezing. This problem occurs during the winter months due to condensation forming inside the piping and valves freezing due to the cold weather and winds. This happens quite frequently, and the only effective way of combating this problem is to erect a hut over and around the valve arrangement located on the roof. This hut serves mainly as a wind break and is usually constructed of wood or sheet metal. Periodic maintenance work is necessary on the valves to keep them free moving. Caution should be used in maintenance work, so as to prevent formation of an explosive mixture of gases in the digester or outside in the immediate area of the roof. It is also imperative that the blow-off pressure set on these valves is correct and checked periodically.

Drip Traps

These are perhaps one of the most important pieces of equipment associated with a gas collecting system. Without properly operating drip traps, all the other gas equipment will become fouled with condensation.

These drip traps are located at the lowest point on the gas collection system and should be subjected to a periodic inspection and clean-out. Excessive condensation can clog a gas system entirely and cause the blow-off to operate. However, it is my opinion that drip traps not properly maintained are indicative that blow-offs are also not being properly maintained, consequently since gas has got to go somewhere, the roof might just lift. There is no mechanical difficulty in the operation of drip traps provided that proper maintenance is given at regular intervals.

Gas meters and flame arrestors will function properly provided that they are protected from condensation through the proper operation of drip traps. A gas meter which does not function properly should be referred back to the manufacturer for remedial action. Gas regulators used on a gas collection and distribution system are subject to periodic rupture of diaphrams, weakening of springs and deterioration of seats and should be subjected in the same manner as drip traps to periodic inspections and clean-ups and possible replacement of parts. A spring loaded gas regulator with diaphram failure will fail closed with slight gas leakage through the vent. This results in gas build-up in the digester. A spring failure will leave the regulator in the open position with a probable vacuum condition resulting in the digester. Seat failure results in a slow leak through the regulator which probably will not create a serious problem. Reversing the seat will usually be sufficient to solve a small leak. Regulators are small pieces of equipment. However they do play an important role.

Waste gas burners are a relatively simple device, the only problem commonly encountered being the failure of the ignition device to operate. Solution of this problem is usually quite simple and ranges from the installation of a wind baffle to acquisition of an independent fuel source.

Piping

Before we leave exterior equipment, I would like to point out one other problem which does exist, not necessarily pertaining to any particular piece of equipment, but to piping in general. Digesting sludge left confined in a pipe, although by accident will quite regularly cause bursting or even more seriously explosion of the confining body due to excessive gas build-up. Solution of this problem is quite simple. Don't leave digesting sludge sitting around in a confined space.

Interior Equipment

We have, up to this point, been discussing equipment which is physically exterior to digesters themselves. I would like now to discuss the equipment which is considered interior to digesters; floating roofs, draft tube mixers and supernatant swing pipes for instance.

Roofs

Roofs on digesters are either fixed or floating. Both are subject to corrosion inside which in severe cases can cause failure of the roof. The only way to prevent failure due to corrosion is to check the underside of the roof, possibly every two to three years to ensure that the corrosion isn't running rampant. This will mean that painting will most likely have to be done under the roof at the time of the inspection.

Floating roof digesters are susceptable to tilting due to unbalance in the roof itself, due to high pressure or high liquid level or both being carried under the roof and due to freezing. Unbalance is easy to remedy with concrete blocks or concrete slabs used as a balast. The problem here is to locate the unbalance. One way of doing this is to raise the liquid level inside the digester to its maximum height and to increase the gas pressure to the maximum allowable. All that has to be done now is to walk around the top of the digester wall, note the location of the gas bubbling around the edge and add balast in this position. Freezing around the outer edge of a floating roof will usually occur when temperatures are consistently below zero. There are several ways to combat this freezing. One is to sprinkle rock salt around the edge of the roof, between the digester wall and the roof skirt. This may cause corrosion if used extensively and is only recommended for infrequent use. Another way which may or may not be effective, depending on the extent of the freezing, is to increase the temperature of your digester. Another way to prevent freezing is to add anti-freeze or fuel oil around the edge. The anti-freeze or fuel oil will naturally float on the surface and in most cases will prevent freezing.

One more problem which occurs with floating roofs is that the wind may tend to rotate the roof thus causing flexing of associated piping and possibly fracture of this pipe. To combat this situation, angle irons should be installed vertically on either side of the rollers to confine them to a vertical motion only. Another problem occurring with floating roofs is that condensation forming in the air space, if allowed to collect, may cause corrosion. Floating roofs are normally designed with first vents to minimize condensation and second with sumps to collect the condensation which does occur. It is well, then, to ensure that the vents are kept free and that the sumps are kept as dry as possible. The sumps should be dewatered either with a stirrup pump (hand operated) or by baling with a pail.

The only equipment which we have not discussed is the equipment which is truly interior to the digester, such as the mixers and supernatant draw-offs, overflow arrangements, etc. Digester mixers are basically of two types -- the propeller type with a draft tube or the gas re-circulation system. Now the basic problem with draft tube mixers is that the digester liquid level may rise to a sufficient height so that it submerges the lower bearing of the mixer. If this is allowed to happen, the bearing burns out and it is then necessary to remove the mixer from the roof and replace the bearing. (NOTE: Narrow operating range of 11" from lower lip to bearing) To prevent this situation from occurring, it is necessary that the bearing is kept well greased at all times and that a watchful eye is kept on the level of the digester. Gas re-circulation systems are subject to plugging of lines and diffusers, slip pipes functioning improperly, and compressors going on the hummer. Normal maintenance and periodic inspections should prevent this from occurring.

Supernatant selection from a digester is normally done through fixed pipes located at various heights on the digester, and it is just a matter of opening a valve to draw off this material. Supernatant swing pipes are also used for this purpose. These pipes are varied in height inside the digester through external controls. The gearings of the swing pipes can seize up inside the digester and/or the pipe itself plug if these pipes are not periodically swung through the arc from their lower limit to their top limit. Slip rings on supernatant transfer wells, overflow wells, etc. also can give problems if not serviced regularly. They may clog with sludge and be the primary cause of other digester malady's.

It is the writer's opinion that a lecture of this type could not possibly be complete without a discussion of perhaps the most difficult problem associated with digester operation, that is, cleaning out of a digester. To illustrate this problem I would like to quote a case history which appears in the "Sewage and Industrial Waste Journal", November 1957. "The digesters of this particular plant are of the fixed roof type with a capacity of approximately 1,000,000 gallons and had been operating satisfactorily for approximately 15 years when overloading of the grit removal equipment in the plant was experienced.

After approximately five years of operation with overloaded grit facilities, it was found that 15% of the digester capacity was occupied by a grit and sand buildup. An initial attempt was made to remove this buildup through the use of a centrifugal dewatering pump connected to the digested sludge suction line. This pump was unable to handle the solids encountered at this point and since this was the only suction line extending into the buildup, the use of this pump was abandoned. Use of the positive displacement piston type pump was then attempted, however, due to low discharge velocity, sand began to settle in the forcemain and eventually

plugged the main. This initial attempt was then given up as a bad job and it was decided that a preplanned program was necessary to effectively clean the digester. Cleaning on a planned basis was then scheduled for approximately one year hence.

During this one year delay, the solids buildup increased to approximately 18% of the digester capacity. The second attempt to clean the digester proceeded as follows:- The digester was shut down completely, however, left in a sealed condition. It was planned that the liquid level of the digester throughout this operation would be kept at a constant level through the use of make-up water. Gas from other digesters was continually re-circulated to a point approximately 10 feet below the surface. This prevented any extreme scum buildup from forming. High pressure gas was also continually applied through flexible hosing to the bottom of the digester, in an attempt to keep the dense material from getting denser. High pressure water was also used in this manner. Since the digester was sealed off, the gas used for the agitation of the digester contents was collected and re-used so that total gas consumption was relatively low. Two pumps with the total capacity of 1500 gpm were used in an attempt to reduce this solid buildup. One pump, a positive displacement type, was connected to the digested sludge draw-off line and the other, a centrifugal type, was connected to the sludge re-circulation suction line. Periodic measurements of the sludge buildup were taken to assess progress of this operation. After 12 hours pumping when approximately 1,000,000 gallons had been removed from the digester, it was found by measurement that displacing the tank contents once over had resulted in removal of about 1/3 of the solids buildup. At this point, it was decided to bring in another pump, a 1000 gpm at 73' head non clog centrifugal pump having torque flow characteristics. The same procedure as noted above was used in the operation of this pump except that the digester level was not kept constant. This level was gradually lowered being governed by the solids content in the pump discharge. Make-up water was gradually reduced until the solids buildup in the pump discharge reached a maximum of 15%. At this point, the make-up water was increased so that discharge of the pump was matched by feed of make-up water. After a few hours of operation with this additional pump, it was found that the suction began to plug and could not be freed by back flushing. Therefore, it was necessary to shut off the gas circulation systems in order to gain access to the digester. High capacity air blowers were then installed on the roof of the digester in order that clean fresh air could be constantly circulated through the digester. A 6" hook on an aluminum pole was then lowered to the pump suction sump and used to free the blockage. As the level in the digester continued to drop, a working platform was erected on a raw sludge feed pipe, and suction blockage removed from this point. This working platform was also used to direct high pressure flushing water in an attempt to dilute the solids build up. The digester was finally pumped to the bottom after approximately 2 days of continuous operation, and use of another 2,000,000 gallons of make-up water. Therefore, approximately three times the digester capacity was used in make-up water to dilute the solids so that pumping was possible. The non-clog pump did not suffer blockage itself at any time, although it

handled up to 15% solids. In this case, the results of solids testing preparatory to cleaning showed a minimum solids content of 12.4% and a maximum solids content of 23%. Mopping up and repair operations were then completed without mishap and the digester returned to service about one week after it was taken out of service.

It should be pointed out from the preceeding, first, that a digester should be able to be cleaned and returned to service in about one week, provided that a well organized program is followed. Secondly, that you can expect to use anywhere from two to five times the digester volume in dilution water.

DIGESTER OPERATION III

by

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An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 24, 1963

DIGESTER OPERATION III

by

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INTRODUCTION

The first course lecture on the Digestion of Sludge covered the subject under the headings, purpose, process, design criteria, operation, and conclusions. The lecture on the second course was titled Digester Operation II and reviewed the operation of single and two stage digester systems.

At this time an example of a process failure in an operating digester will be reviewed. Coverage will be given to the steps taken to return the digestion process to a normal alkaline digestion state.

A review will be made of tests required to control the digester process. Note will be made of the minimum tests required at a small plant and also of that required for a large operation.

FAILURE AND RECOVERY OF DIGESTION PROCESS

The example outlined in this lecture occured at an activated sludge sewage treatment plant serving a city having a population of 20,000 persons. The raw sewage contained a high percentage of industrial waste waters mainly from a meat packing plant. The necessary sewage treatment plant detail is presented in tabular form.

Plant Loadings During 1961

Average flow to plant	2.62 mgd
Average BOD of plant influent	311 ppm
Average BOD loading	8150 # per day
Average BOD removal by plant	7600 # per day
Average suspended solids in plant influent	267 ppm
Average suspended solids loading	7000 # per day
Average suspended solids removed by plant	6300 # per day

Calculated volatile solid loading to digester	4200 # per day
BOD population equivalent	48000 persons
Actual population	20000 persons

Digester Detail

Digester liquid capacity

1st stage	67,000 cu. ft.
2nd stage	33,000 cu. ft.
total	100,000 cu. ft.

Mixing, gas recirculation, 30 cfm compressors

Boiler capacity 432,000 BTU's per hr.

Digester Loading

	Actual	Recommended
Cu. ft. of digester capacity per capita		
1st stage	3.3 cu. ft.	3-6 cu. ft.
complete	5.0 cu. ft.	
Pounds of solids to first stage digester per month	3.0	2-3
Cubic feet of digester volume per pound of volatile solids per day to the first stage digester	16	25

Most digester installations are designed with a generous safety factor. The volume in a digester usually exceeds the need and if adequate mixing and temperatures are supplied minor overloading will not cause a process failure. With this example the actual loading approaches recognized design factors. However, normal loading does not approach that allowed in a high rate design.

The table titled operating data, appended as Item I and II, contains information on the volatile acids level, raw sludge loading, pH, gas production and operation. During the period under review the digester temperature was maintained at 90°F. and adequate mixing was carried out.

Process Failure

On November 25th, unknown to the plant operator, a load of ether soluable material was discharged to the tributary sewer system. It is believed that the resultant heavy load on the digester system caused an increase in the volatile acids level and finally a process failure. The volatile acids tests failed to indicate the impending process failure and the first major sign was a reduction in the gas production. Also, the burning quality of the gas made it unsatisfactory for boiler use. The volatile acids test was investigated on December 4th, and found to be in error.

With the use of new reagents the volatile acid level was found to be 3,280 ppm. This figure is much above both the average operation range of 200 - 500 ppm., and the much quoted maximum allowable of 2,000 ppm. It is noted that digestion may take place at volatile acids levels of 3,000 ppm but many factors such as ion toxicity and pH will determine the recovery rate.

Plan of Process Recovery

When a digestion process failure has occured; indicated by low gas productions, poor quality gas, and an elevated volatile acids level; the operator must organize a recovery programme. In setting up this programme the most important step is that of ascertaining the reason for the process failure. There is no use in spending time and moneys in re-establishing an alkaline anaerobic digestion process if the failure is to recur at a later date. The process failure can be caused by one or a combination of the following reasons:

1. overloading
2. inadequate temperatures
3. inadequate mixing, and/or
4. metal ion or chemical toxicity.

In the example under review the probable cause for process failure was short term overloading. The offender could be cautioned and asked not to batch discharge excess quantities of organic material to the sewer system. Possibly an interceptor grease trap is required on the company's outfall sewer.

After the cause of the failure is thought to be known and corrected, a plan of recovery should be outlined. Possible recovery programmes are:

1. the removal of all the liquid and sludge from digester and the restart of a new digestion process;

2. the removal of part of digester contents for the purpose of reducing the volatile acids level by dilution and at the same time the raw sludge loading is reduced or completely stopped;

3. the addition of lime for pH adjustment while reducing or eliminating further raw sludge loading; or

4. the reduction of the raw sludge loading to the digester.

1. Remove Sludge from Digester

This step is drastic and it may require a period up to two months to re-establish active digestion. The disposal of

sour sludge from a digester is a major job in itself and then seed sludge must be obtained to speed the digester start-up process. In meantime a large portion of the raw sludge must be hauled for farm or other disposal. On the other hand, the process failure may be so complete that ineffective half hearted solutions will only waste time.

2. Removal of Part of Digester Content

The dilution of the sour digester contents with raw sewage, not sludge, will reduce the volatile acids level. In using this method of solution active digestion agents will remain. Again the raw sludge is redirected to another method of disposal and should not be returned to the digester until the volatile acids level is below 1,500 - 1,000 ppm. Full loading is not attempted until the volatile acids level is reduced below 750 ppm.

Below a given volatile acids level the recovery process will be dramatic while above 2,000 ppm progress may be slow. In the 1,000 to 1,500 ppm range very active digestion may occur and unless adequate mixing is provided foaming can result.

3. pH Adjustment

In the given example the pH did not drop below 6.8, therefore pH adjustment would not aid in improving the digestion process. Also pH adjustment could cause an ion toxicity problem.

When the pH is below 6.8 some adjustment is in order but care should be taken that the true cause of the failure is first eliminated. As in the dilution method outlined above raw sludge loading must be discontinued.

4. Reduced Loading

When the digestion process failure is not complete the most satisfactory method of recovery consists of the complete or part removal of the raw sludge load. As previously stated the reloading is commenced when the volatile acids level drops below 1,000 - 1,500 ppm. Complete reloading should wait until the volatile acids level is below 750 ppm.

Review of Example

In the example under consideration it was believed that the failure was caused by overloading. When the gas would not burn in the boiler sludge pumping was discontinued. The pH was satisfactory at 6.8 to 7.1 so lime was not added. As gas

was being created and the volatile acids level did not rise above 3,300 ppm it was decided to follow the reduced loading technique. This approach appears to have been effective and a complete process recovery was achieved in approximately twenty days. The recovery programme varied slightly from the idea and raw sludge was pumped to the digester in small quantities before the volatile acids concentrations reached the 1,000 to 1,500 ppm level. This action would tend to delay recovery. On the other hand the reloading programme might have been accelerating when the volatile acids level dropped below 750 ppm. The full load return should not be attempted in one day.

The process recovery achieved at this plant was well planned and carried out. The operator had the benefit of:

(i) a holding lagoon isolated from housing where excess sludge could be directed during period of process failure;

(ii) good laboratory facilities and the time and ability to carry out the difficult volatile acids and other tests;

(iii) a digester system with adequate mixing so that the scum blanket and inactive process areas could be kept to a minimum;

(iv) adequate boiler facilities which enable satisfactory temperatures in the digester.

On the other hand the two stage digester system at this plant is constructed in one unit which prevents flexible operation. In a plant having separate digesters the raw sludge can be directed to the second stage digester while the first stage unit is rested and returned to satisfactory condition. Excess raw sludge had to be removed from the holding pond first thing in the spring or an odour problem could have been created.

CONCLUSIONS

The example reviewed pointed out the need for accurate laboratory control over the alkaline anaerobic digestion process as used for sludge reduction and stabilization in a sewage treatment plant. The other alternative is to have complete control over:

BOD loading
mixing
heating
industrial wastes

The example under review confirmed the premise that the pH test is of little use in forcasting process failure.

In a small plant the loading factor could be reduced thereby offsetting the effect of a part alteration of one of the four affecting factors. When a town does not contain chemical or metal plating industry the BOD loading is the only instantaneous unknown. The heating and mixing functions are of course effectively measurable.

In a large plant where high loading factors are used the minimum required laboratory tests for digester operation are:

volatile acids
alkalinity
pH
gas composition
raw sludge dry solids, total and volatile
supernatent dry solids and BOD
digested sludge dry solid, total and volatile.

In a smaller plant lacking laboratory equipment and having an oversized digestion system the volatile acids and other tests will have to be carried out at a central laboratory. The plant itself can keep accurate data on sludge pumping and wasting, mixing schedules, and heating.

With the knowledge we have of the alkaline anaerobic digestion process there should be few unforcast process failures. Also, if a failure does occur we should be able to plan an effective recovery programme. This programme may prove to be costly but half measures will only lead to failures and further expenses.

Operating Data
Item I
Covering Period of Digestion Process Failure and Recovery

Date	Volatile Acids in p.p.m.	Raw Sludge Loading in Gallons	pH	Digester Gas Production in Cubic Feet	Comments
Normal Operation	200-500	7,000	6.8-7.2	20,000 -	
Nov. 25 to Dec. 4	641-1,086*	-	6.8-7.1	-	load of ether soluable material released to sewers on Nov. 27
Nov. 27	-	-	-	9,211	gas would not burn in boiler raw sludge pumping discontinued
Dec. 4	3,280	1,135	-	1,290	*reagents used in volatile acids test found to be unsatisfactory therefore previous tests not accurate
Dec. 5	3,192	1,000	7.2	902	
Dec. 6	3,226	1,021	7.2	1,419	
Dec. 7	3,381	1,000	7.5	5,002	
Dec. 8	3,385	1,154	7.4	9,210	gas would burn in boiler but heat value low and boiler water temperature dropped 20°
Dec. 9	-	1,000	-	14,064	
Dec. 10	-	1,000	-	17,101	
Dec. 11	1,899	1,015	-	16,065	gas used in boiler and heating value had returned

Item II
Operating Data Continued

Dec. 12	1,596	2,000	-	15,649	raw sludge pumping doubled to 2,000 G.P.D.
Dec. 13	1,407	2,075	-	15,342	
Dec. 14	1,047	2,002	7.4	14,908	
Dec. 15	-	2,002	-	12,730	sludge removed from digester for one hour
Dec. 16	-	3,000	-	13,840	
Dec. 17	-	3,533	-	12,297	
Dec. 18	704	3,950	7.9	-	digested sludge removed for 1.25 hours
Dec. 19	704	3,375	7.9	15,350	digested sludge removed for 2.30 hours
Dec. 20	463	3,825	7.8	17,515	
Dec. 21	240	4,500	7.2	16,709	
Dec. 22	-	6,200			
Dec. 23	292	-	7.0	21,472	all raw sludge pumped to digester
Dec. 24 to Dec. 28	200-300	-	7.0-7.2	16,000-22,000	gas burns well yellow orange flame

HEAT EXCHANGERS

by

R. J. Norton

Division of Plant Operations - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 24, 1963

HEAT EXCHANGERS

by

R. J. NORTON

Plant Operations

There are a number of different types of heat exchangers. One type is a combination of boiler and heat exchanger built as a single unit; the two hot water compartments are divided by a centre wall. In some models the water flow from the boiler side to the exchanger side is controlled by a thermal valve with a handle or handles on the outside wall which either closes or opens the valve. This type of control is manual.

Other combined units have a water pump which circulates the water from one side to the other and return and is controlled by a thermostat.

Located in the exchanger side there are a number of tubes running horizontally from top to bottom through which the sludge is circulated from the digester and return thereby bringing the digester contents up to and maintaining an operational temperature. This sludge recirculation is also controlled by a thermostat.

Other types of heat exchangers are a single unit frequently located in another room and sometimes in another building from where the boiler is located. The hot water is circulated by pump from the boiler to the exchanger and back again as in a combined unit.

Of the single units there are two outstanding types, a rectangular and a round type. The rectangular type of exchanger is called a concentric type of water bath. The sludge tubes are incased in another tube containing the hot water. This type affords a more concentrated heat transfer from water to sludge and has less heat loss from the water being in contact with the outer wall surface of the unit. The water flow is in the opposite direction to that of the sludge.

The round exchanger is much smaller in size to the rectangular types. The inside construction is not pipes but two concentric channels back to back spiralling from the top

to the centre of the exchanger. The circulating sludge enters at the top, spirals through to the centre and out. The hot water enters at the centre and spirals to the top and out, giving a counter current flow pattern. Heat exchangers usually operate in a heat range of 130°F. minimum and 180°F. maximum.

HEAT EXCHANGER MAINTENANCE

All heat exchangers require maintenance and servicing to keep them in top operating efficiency. This servicing must be done on a regular basis generally once or twice per year.

When the exchanger has been emptied and the sludge tube bends have been removed from both ends, the tubes should be inspected with a light for baking or sticking sludge on the sides of the tubes. After cleaning the tubes, they should be checked for pitting and wear; a notation should be made on the maintenance card as to the general condition of the tubes, the number and location of any tube considered worn enough for replacement. Replacement tubes could be then ordered and be on hand for installation during the next inspection.

The hot water tubes in a concentric type exchanger should also be inspected at the same time for accumulations of lime and/or rust deposits and general condition. Check mounting rods for straightness, clean threads with a wire brush to ensure even tightness of mounting nuts. The tools for cleaning the sludge tubes are a wire tube brush usually supplied with the machine, a rag swab sometimes used for further cleaning. Always follow through beyond the end of the tube before redrawing it the other way.

HEAT EXCHANGER OPERATION

The operation of the boiler unit, whether it is combined with or separate from the exchanger, is the same. They usually operate on a digester gas and natural gas combination or a digester gas and oil combination. The selection of fuel to be used in either combination is achieved by adjustments on the pressure regulating valves located on the machine.

To illustrate how the heat exchanger uses the gas produced by the digester, it is necessary to start at the digester itself. The main gas produced during the satisfactory process of digestion is called methane and is formed of carbon and hydrogen. This gas is used as the fuel in the heat exchanger in preference to any other types of fuel, since it costs nothing. Most small plants provide only gas storage under the domed digester roof. Normally, all the gas that is produced is used up immediately either by the heat exchanger or the waste gas burner. Therefore, no gas storage is necessary.

The gas is produced at a fairly even rate in the digester and rises to the top and collects under the roof. It is maintained here at a certain pressure dependent upon the pressure regulating valve in the waste gas burner line. Assuming a pressure equivalent to 8" of water is required to operate the heat exchanger, then the waste gas pressure regulating valve is set to remain closed until 8" of pressure is reached in the service line. The pressure relief valve on the digester roof is then set at 9" or one inch greater than the pressure setting of the waste gas regulator. All gas, therefore, under pressure greater than 8" is by-passed or released to the waste gas burner.

If the pressure under the roof should suddenly rise or the pressure regulating valve stick shut, then the pressure relief valve would permit the escape of gas in excess of 9" of pressure, which would be held under the roof. When the heat exchanger is using the digester gas and natural gas combination, the afore-mentioned pressure regulating valves on the machine are used to control the pressure of each supply and the pressure of the main feed to the burner. These valves are set so that any pressure drop below the lowest setting for digester gas will activate the valve on the digester gas line closing it and opening the valve on the natural gas line, thereby switching to natural gas firing. This normally occurs without interrupting the burner operation. Similarly, in the case of a heat exchanger operating on digester gas and oil combination, the occurence of a pressure drop in the digester gas line activates the pressure regulating valve which throws a mercury switch starting the oil feed pump and switching to oil firing. Again, heat exchanger operation can proceed uninterrupted during such a change-over

When the digester gas returns to a normal 8" pressure, the pressure regulating valves on the heat exchanger again comes into service. The digester gas regulating valve opens and the natural gas valve closes, or the valve controlling the mercury switch closes shutting off the oil feed pump switching back to digester gas firing. During this transfer of fuel supply, the heat exchanger operation continues uninterrupted.

GAS COLLECTING AND CONTROL UNIT MAINTENANCE

The gas service piping and the waste gas piping should be kept drained of water at all times.

Pressure regulating valves on the heat exchanger and the waste gas line should be stripped down on a regular basis and checked for pin holes in the diaphragm, sticky valve action and accumulation of moisture and sludge deposits. Flame arrestors should also be dismantled on a regular schedule, cleaned and checked for plugged cones, sludge accumulation, and in some types sticky valve action. Flame arrestors and regulating valves should have the plugs removed from the bottom of the casing periodically to drain out the water.

Pressure relief valves and vacuum relief valves on the digester roof should be inspected regularly especially during winter in case of freezing.

A regular inspection and maintenance schedule will not only ensure a trouble-free operation but will increase the length of operating time between take-downs and repairs to the gas meters. It also lengthens the operating life and efficiency of a gas blower and other gas equipment when gas recirculation is used for digester mixing.

BOILER MAINTENANCE

The fire tubes should be inspected frequently and cleaned as often as needed. Partially plugged tubes give improper combustion, and a loss of heat transfer to the water. When cleaning the fire tubes, follow through with the tube brush beyond the end of the tube. This ensures that all the soot is pushed clear of the tube but also if the brush is reversed while in the tube, a number of the bristles are pulled out each time, reducing the life of the brush and producing a poorer cleaning job. When cleaning out the fire box inspect and replace any broken fire clay brick.

SIGNIFICANCE AND IDENTIFICATION OF PROTOZOA IN ACTIVATED SLUDGE

by

C. F. Schenk and C. J. Howes

Division of Laboratories - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 24, 1963

SIGNIFICANCE AND IDENTIFICATION

OF PROTOZOA IN ACTIVATED SLUDGE

by

C. F. SCHENK and C. J. HOWES

INTRODUCTION

In order for you to gain some understanding of the significance of different types of Protozoa throughout activated sludge systems, a knowledge of the function of Protozoa in activated sludge, and of the means by which they are studied and identified, are essential prerequisites. We hope to make some progress in completing this first requirement within the next hour and to subsequently make a start in our laboratory to acquaint you with microscopic equipment and at least some of the more common types of Protozoa which are found in activated sludge.

MAJOR GROUPS OF PROTOZOA

The Protozoa are single-celled animals which are generally distinguished from unicellular plants by the fact that most of the latter contain the pigment chlorophyll, which gives them a green colour. This substance permits the production of food from inorganic materials within plants in the presence of sunlight, which process is called photosynthesis. Unlike the unicellular plants which manufacture their own food, photosynthesis does not take place in the Protozoa and other animals. Protozoa feed upon solid food of organic origin (either alive or previously alive). While there are a few intermediate forms whose correct classification is disputed among different microbiologists, for our purposes it will be satisfactory to consider the Protozoa as non-pigmented unicellular animals, most of which metabolize solid food.

For the sake of simplicity and since we are not particularly interested in taxonomic details related to the Protozoa, we can divide them into five major groups:

(1) Ciliata - have cilia for movement and capturing food e.g. Paramecium
(2) Mastigophora - have flagella for movement e.g. Bodo

(3) Sarcodina - move and capture food by extending pseudopods e.g. Amoeba
(4) Sporozoa - parasites in plants and animals
(5) Suctoria - bear tentacles for capturing prey.

Only three of these groups, the Ciliata, the Mastigophora and the Sarcodina play important roles in the activated sludge process, although members of the Suctoria group are occasionally observed.

The Ciliata

These animals have short, hair-like appendages called cilia which may extend from the animal's body around its entire margin or which may only be present at the anterior end. Cilia are utilized in movement and those near the entrance to the gullet for capturing food. Possession of cilia is common to all members of this group, although they are more readily apparent in some species than others. The Ciliata group is comprised of both free-swimming and attached, stalked forms.

Mastigophora

The members of this group bear flagella, whip-like appendages which enable the organism to move through the water with a corkscrew-like motion. One or more flagella may be present (singular - flagellum).

Sarcodina

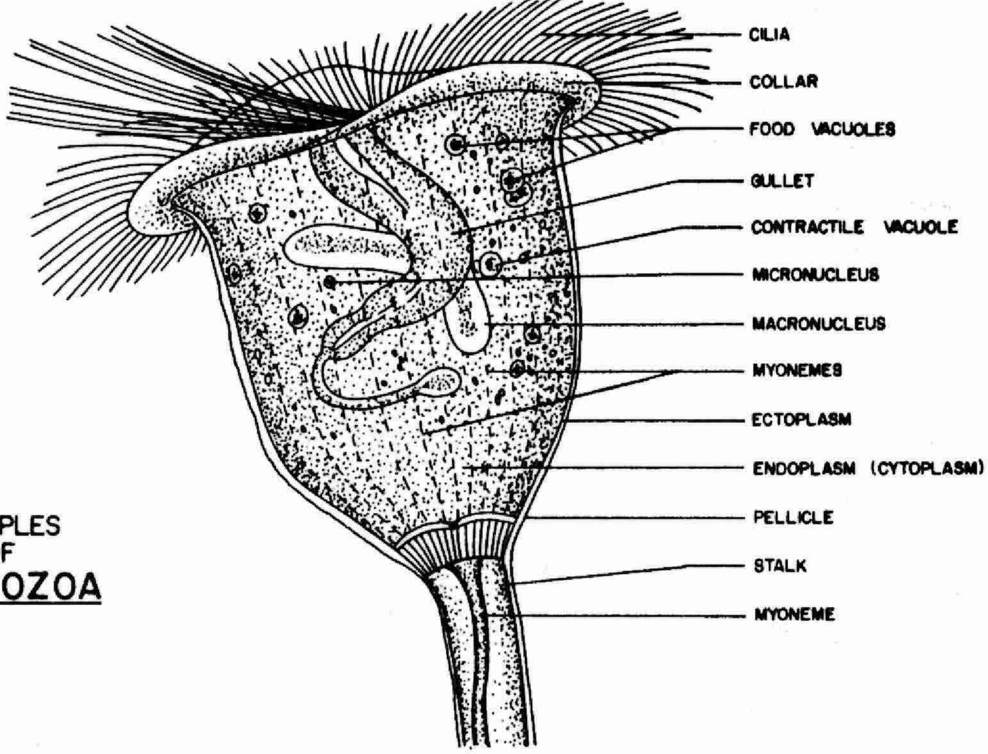
The presence of pseudopodia (false feet) is the distinguishing feature of members of this group. These tiny animals have an extremely plastic cell membrane and the fluid protoplasm within the cell literally "flows" in the direction that the animal wishes to move. Protoplasm is the clear, jelly-like substance common to all plant and animal cells in which the basic life processes function. Pseudopodia may also be extended to engulf food particles which are assimilated through the cell membrane.

Terminology Used in the Description of Protozoa

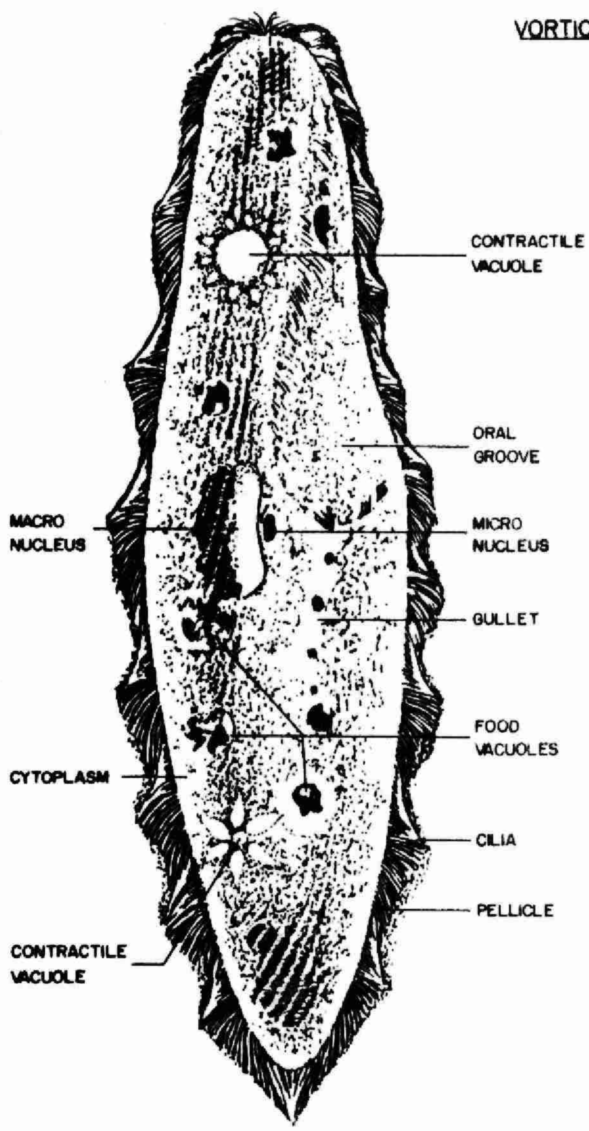
Pellicle - the cell membrane which determines the shape of the animal and encloses the inner components of the cell.

Protoplasm - the jelly-like substance which is contained by the pellicle in which the animal's life processes function. The protoplasm is divided into a clear outer portion called the ectoplasm, and a granular inner mass, the endoplasm.

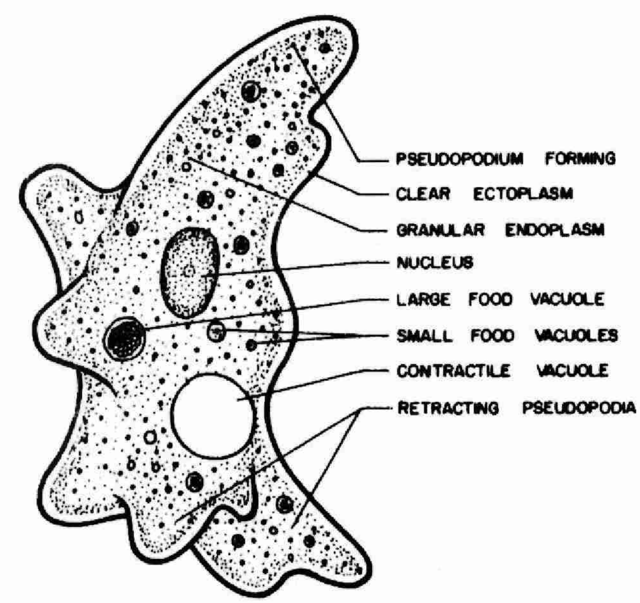
EXAMPLES OF **PROTOZOA**



VORTICELLA - AN ATTACHED CILIATE



PARAMECIUM - A FREE-SWIMMING CILIATE



AMOEBA - REPRESENTS THE SARCODINA

OWRC MAY 1963 DRN. BY : O.F.

Macronucleus - the larger nucleus which governs bodily activities within the cell. Not present in Sarcodina

Micronucleus - the smaller nucleus which is involved in reproduction.

Contractile Vacuole - a relatively large, clear structure which is responsible for gathering and excreting water from the cell.

Food Vacuoles - structures in which food is being broken down by enzymic action.

Oral Groove - present in ciliates - opening lined with cilia into which food particles are drawn.

ROLE OF PROTOZOA

It is only within the last thirty years that there has been a gradual acceptance that the stabilization of liquid organic wastes by the activated sludge method is primarily a biological process. Bacteria are responsible for stabilizing the organic matter by utilizing it as food to permit growth and reproduction. They are able to secrete extracellular enzymes which change the organic matter from an insoluble to a soluble state. Floc production also results from bacterial activity which enhances the settlability of the organic sludge. The types of bacteria which prevail in any aeration system vary, depending upon the nature of the wastes being treated.

The Protozoa remove excess bacteria from the sludge, thereby stimulating optimum bacterial development and assisting in BOD removal and in reducing turbidity. Non-living organic material is consumed in addition to the bacteria.

Some general remarks can be made with respect to interpreting the character of activated sludge from the types of Protozoa present. If sludge is in good condition there will be few flagellates, Amoebas and other rhizopods, and ciliates such as Carchesium, Vorticella, Espistylis (all attached forms) and free-swimming forms such as Aspidisca and Colpidium will predominate. Where the sludge is unsatisfactory, higher numbers of flagellates and rhizopods will be present. McKinney and Gram (1956) discuss the progression which takes place in an activated sludge system. They point out that the flagellates are unable to compete with the smaller bacteria and make a brief appearance before numbers of bacteria increase sharply. The free-swimming ciliates on the other hand, successfully feed on the bacteria as the bacterial population increases and contribute to reductions in turbidity and BOD. As the numbers of bacteria are reduced below the demands of the energy-consuming free-swimming ciliates, the stalked ciliates increase in numbers because they have a lower energy requirement.

While hasty conclusions should not be drawn, the kinds of Protozoa present may be used as a reasonably accurate index of the efficiency of treatment.

ANIMALS OTHER THAN PROTOZOA OCCASIONALLY OBSERVED

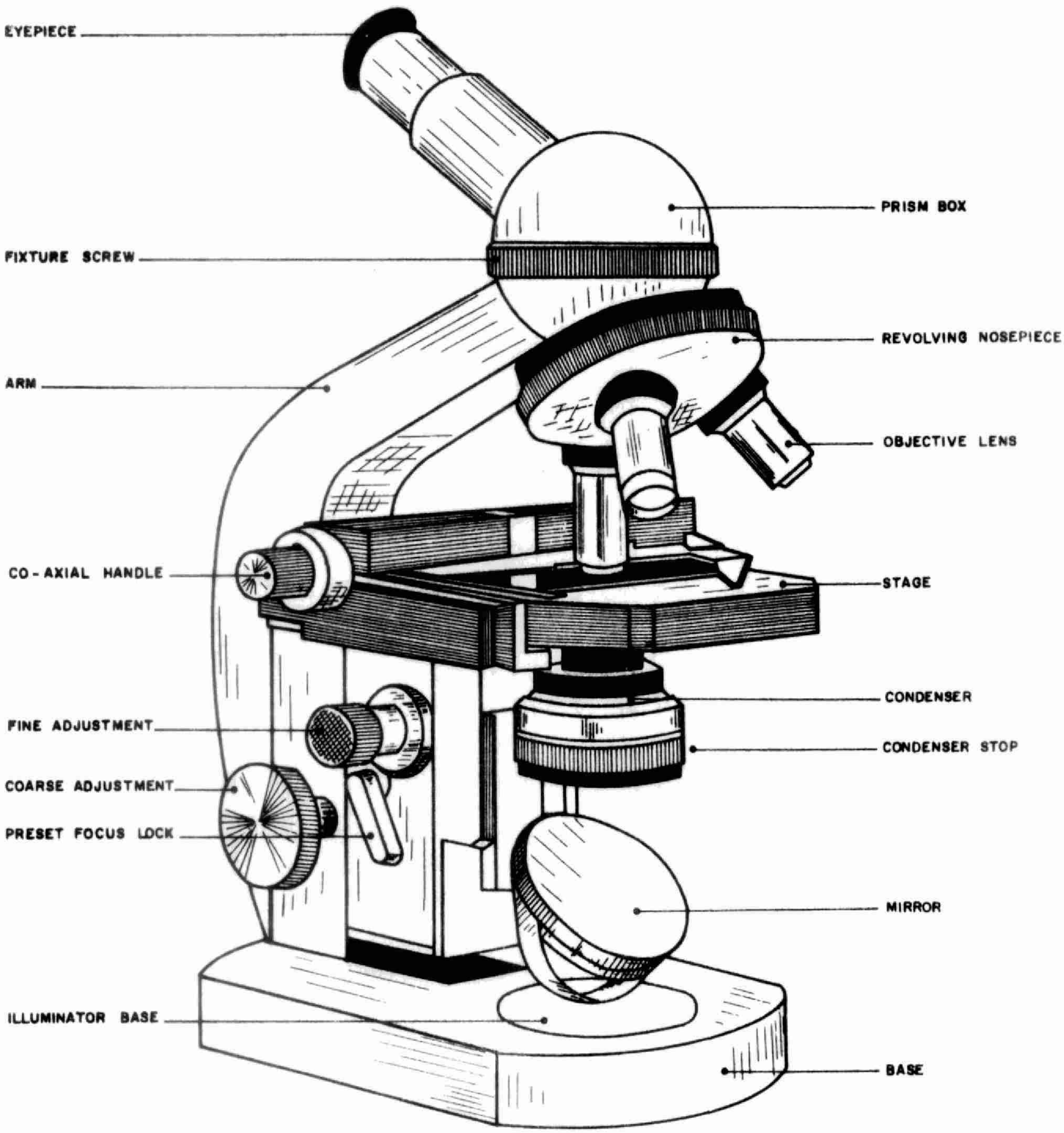
Occasionally, more complex multicellular animals will be observed in activated sludge which are not generally considered to play as important a role in the stabilization process as the Protozoa. The rotifers are the members of this group which probably have greatest significance since they are fairly common in good quality sludge and do assist the ciliates in removing organic material and bacteria. They are characterized by the ring or rings of cilia at the anterior end of the body and by the powerful internal grinding jaws which are clearly evident. Tiny nematodes or roundworms are occasionally found and larval forms of midges and other winged insects may be present also.

USING THE MICROSCOPE

The diagram which has been included indicates the various parts of the compound microscope which one must use to examine activated sludge. The compound microscope has two separate lens systems. The one nearest the specimen, called the objective, magnifies the specimen a certain amount. The eyepiece, which is the second lens system, further magnifies the image produced by the objective. Three different objectives are provided on a revolving nosepiece so that high and low magnifications may be obtained. Usually the standard objectives are 10X, 43X and 97X. In examining Protozoa, a 10X eyepiece is usually used in combination with a 10X objective so that a magnification of 100X is obtained.

Glass slides and coverslips are essential in addition to the microscope. A drop of the liquid to be examined is placed on a glass slide and covered with the smaller coverslip. The coverslip reduces the depth of the water on the slide so that the organisms will generally be in focus and prevents the objective lens from coming into contact with the sample.

When examining a slide which has been prepared as just described, the slide is placed under the low power objective and the coarse adjustment knob is turned until the objective is about an eight of an inch from the coverslip. One should then look into the eyepiece, and with microscopes having a moveable stage, the stage should be lowered with the coarse adjustment knob until the organisms and sludge constituents come clearly into view. Fine focusing can then be achieved with the fine adjustment knob. When the high powered objectives are used, the coarse adjustment knob cannot be turned slowly enough when the object comes close to being in focus and one must use


EYEPIECE
PRISM BOX
FIXTURE SCREW
REVOLVING NOSEPIECE
ARM
OBJECTIVE LENS
CO-AXIAL HANDLE
STAGE
CONDENSER
FINE ADJUSTMENT
CONDENSER STOP
COARSE ADJUSTMENT
PRESET FOCUS LOCK
MIRROR
ILLUMINATOR BASE
BASE

the fine adjustment to obtain a sharp image. If there is too much light and resulting glare, the disc diaphragm should be turned under the microscope stage to reduce the size of the diaphragm opening. It is essential to remember that the image formed by the compound microscope is inverted; the object is seen upside down and reversed so that the right side is at the left.

REFERENCE

McKinney, Ross E. and Andrew Gram. 1956. Sewage and Industrial Wastes 28, 1219.

LABORATORY PERIOD - PROTOZOA IN ACTIVATED SLUDGE

1. The Compound Microscope

   1) Note that image is opposite and upside down.
   2) Practice focusing - course adjustment and fine adjustment
   3) Moving the slide to different positions - co-axial handle.

2. Preparing the Slide

   1) Place single drop of sample at centre of slide.
   2) Carefully place coverslip over the drop so that air is expelled as the coverslip is lowered.

3. Examining Living Materials

   1) Amoeba - note nucleus, contractile vacuole, food vacuole, pseudopodia.
   2) Paramecium - note cilia, oral groove, pellicle, macronucleus, micronucleus, food vacuoles, contractile vacuoles.
   3) Vorticella - note cilia, macronucleus, micronucleus, food vacuoles, contractile vacuole, single contractile stalk.
   4) Epistylis - note features as in Vorticella, but note the branched stalk.
   5) Carchesium - similar to Epistylis.

4. Examination of Activated Sludge

   Check a sample of the activated sludge, noting and listing as many of the organisms as you are able to identify with the help of the assistants present.

INDUSTRIAL WASTES III

by

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An Address To
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Senior Sewage Works Operators' Course
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April 25, 1963

INDUSTRIAL WASTES 111

by

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The two previous lectures dealing with the treatment of industrial wastes in municipal sewage treatment plants outlined the characteristics of the most commonly encountered wastes, the difficulties that are to be expected as a result of their uncontrolled discharge to the municipal sewers, and the controls or restrictions that should be imposed by the municipality to protect the sewerage facilities and to prevent upset or interruption of the treatment processes. It is readily apparent in practice, however, that it is one thing to establish a set of control limits and quite another thing to make them completely effective. Even where controls are in effect, the sewage treatment plant operator is often faced with the task of having to treat wastes that either cannot be practically controlled at the source to conform to the by-law limits, or reach the sewers in excessive concentrations due to accidental spills or other unexpected discharges. It is small comfort to the operator to know that a sewer-use by-law is in effect, when he gets a slug of oil or other objectionable waste from any one of a number of possible sources.

To go back to the basis for setting out control limits in an industrial waste by-law, domestic sewage is fairly uniform in composition from one municipality to another, and treatment methods have been well standardized in that knowledge. Industrial wastes, on the other hand, vary widely in both composition and quantity, and it is the purpose of establishing limits for their acceptance into the municipal sewers to ensure that the facilities that were basically designed to treat so-called "normal" sanitary sewage will function equally well with the addition of industrial waste This means that wastes that lend themselves to treatment by the conventional processes are controlled so as not to exceed the capabilities of the treatment plant to treat them, and that those wasts which do not benefit from treatment, or are toxic or otherwise harmful to the process, are either kept out of the system or minimized so that they are effectively diluted in the combined sewage mixture.

Recent efforts to bring about control of industrial wastes to conform to the recommended limits have shown that most industries can comply without too much difficulty, but there are usually a few, which, by reason of their location or their inability to finance the necessary pre-treatment facilities, cannot do so. Rather than compel such industries to undertake measures that would threaten their ability to remain competitive with other similar industries, it is often necessary for the municipality to accept the wastes into the municipal system without the desired control. This concession can only be made if the wastes are amenable to conventional treatment, and should specify surcharges that would off-set at least a portion of the extra costs of treatment that are incurred.

In evaluating the industrial waste loading on any one treatment plant, it is therefore not only a matter of determining which wastes can be treated, but how much of each can be safely handled. With the exception of extremely dangerous explosive or toxic materials, and substances that lead to deterioration or blockage of the sewers, most industrial wastes lend themselves to treatment in municipal plants, or are effectively removed from the sewage without adversely affecting the plant or process. If the quantity and characteristics of the industrial wastes are known, most sewage treatment plants can be operated to accomodate loadings that do not conform to the recommended limits. The following discussion deals briefly with some of the over-loading to be expected.

The activated sludge plant design is commonly based on a loading in the aeration tanks of 35 pounds of applied BOD per 1000 cu. ft. of aeration tank capacity. Loadings in the order of 50 pounds of applied BOD per 1000 cu. ft. are possible by modifications of conventional aeration diffusers, and by the use of special aeration devices, loadings up to 400 pounds of applied BOD per 1000 cu. ft. can be handled. This means that packinghouse wastes, for example, can be given secondary treatment well above the BOD concentration indicated by the industrial waste by-law, as long as the pattern of BOD loading can be determined and the necessary modifications made in aeration to conform with it.

In considering modifications to provide for increased BOD loading, consideration should also be given to the <u>rate</u> of oxidation of industrial wastes. Any measurement which only takes into account the total oxygen requirements may be misleading because of the wide variation in the rate of oxidation from one type of industrial waste to another.

Also important in the design and operation of activated plants for treating combined wastes is the return sludge facility. As the BOD concentration of the combined waste increases the return activated sludge-waste ratio should increase. It is customary to operate on normal sewage strengths

with a return sludge-sewage ratio of 1:4. When loadings are increased by the discharge of strong wastes such as can be expected from packinghouses, tanneries, canneries, etc., a ratio of 1:2 is often required, and, under extreme loadings, a ratio of 1:1 can be used.

Another problem that is often encountered in treating industrial wastes in municipal systems is putrefaction that occurs in the sewers and the primary section of the treatment plant. Some wastes (including those from tanneries, packinghouses, dairies, textile plants, etc.) decompose very rapidly when mixed with sanitary sewage, particularly when they are first held in flow-equalizing facilities at the industry prior to discharge to the sewers. The replacement of aerobic by anaerobic conditions is accompanied by the typical gas-forming decomposition which not only gives rise to an odour problem at the sewage treatment plant but also changes the settling characteristics of the primary sludge and prevents good solids removal. Pre-aeration only serves to heighten the odour problem, although it might relieve plant operating difficulties. Pre-chlorination can be used effectively, but, if the condition is chronic, correction should be made at the source by aeration in the industrial waste equalizing facilities, or by chlorination or other suitable chemical treatment prior to discharge to the municipal sewer. Even where wastes from such industries are received in an aerobic condition, care should be taken to keep up with the removal of sludge as it accumulates to avoid putrefaction occurring in the primary tanks. If the solids loading is excessive, as it may be at times from a tannery, packinghouse or cannery, provision for hauling away undigested sludge may have to be made to avoid over-taxing the digester.

One of the greatest difficulties encountered in the combined treatment of domestic sewage and industrial wastes is related to radical changes which may occur in the characteristics of industrial wastes produced through change of processes or the development of new products. This matter is of such importance in industries which provide their own treatment facilities that many of them have an established policy that no major changes can be made in processing until the effect of the wastes on the treatment process can be evaluated. A similar policy should apply in all cases of combined municipal-industrial treatment plants, and to new industries with liquid wastes that may wish to discharge to a municipal sewerage system.

A most important point in treating industrial wastes is knowing the sources and establishing an understanding with the industries of the need for control. Many people in industry have little knowledge of what happens to their wastes after they are sewered, and it is often useful to ask them to tour the treatment plant so that they may appreciate the problems at hand. This may not serve to solve difficult problems, but at least industry might be persuaded to give warning of accidental spills or discharges so that preventive measures can be taken before the treatment process is too seriously affected.

ANALYSIS AND CRITERIA TO MEASURE STRENGTH OF INDUSTRIAL WASTES

by

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An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 25, 1963

ANALYSIS AND CRITERIA TO MEASURE STRENGTH OF INDUSTRIAL WASTES

by

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As the majority of you recognize many of the problems in the design and operation of municipal sewage treatment plants arise not because of the nature of the domestic sewage load but because of the peculiarities of the various industrial wastes that are combined with. Therefore, in this lecture I propose to set out the criteria to be examined and investigated when wastes are under consideration for discharge to the public sewer or are already being discharged but presenting problems. The criteria will be broken down into three major divisions physical chemical and biological. Table I outlines these divisions and gives the sub-headings in each.

TABLE I

Physical Criteria

(a) Volume
(b) Temperature
(c) Residue on Evaporation Fixed and Volatile
(d) Suspended and Dissolved Solids
(e) Settleable Solids
(f) Coarse and Floating Solids
(g) Oils and Greases

Chemical Criteria

(a) pH, Acidity and Alkalinity
(b) Colour
(c) Taste and Odour
(d) Radioactivity
(e) Synthetic Detergents
(f) Dissolved Substances

Biological Criteria

(a) Dissolved Oxygen
(b) BOD
(c) Other Oxygen Consumed Tests
(d) Toxicity
(e) Nitrogen Compounds
(f) Direct Uptake of Oxygen

PHYSICAL CRITERIA

(a) Volume

A waste characteristic of major importance in disposal is total waste quantity or volume. Just as the dilution by the receiving stream becomes less as the treated effluent increases, dilution by sewage for a given flow of industrial waste will improve if the waste volume is kept to a minimum.

At the sewage plant itself good control over waste volumes entering the sewers will show in:

(1) reduced chemical costs for chlorine, coagulating aids, filter aids
(2) improved settling time in the primary section
(3) lower capital costs and longer plant life for the sewage treatment plant

(b) Temperature

Abnormally hot waters occur commonly from the textile, laundry and chemical industries, and should not be discharged to the sewers with a temperature higher than 150°F. Recovery of waste heat can result in fuel saving at the industry and prevent accelerated corrosion, thermal stresses and breakdown of jointing material in the sewers. Hot wastes can also lead to more rapid decomposition of organic matter leading to the development of septic conditions in long sewers and can interfere with primary settling operations at the sewage treatment plant.

Other physical analyses which can be preformed on industrial wastes to indicate their nature are:

(c) Residue on Evaporation
(d) Suspended and Dissolved Solids
(e) Settleable Solids
(f) Coarse and Floating Solids

With the residue on evapouration test it is the nature of the material remaining that is as important as ignition of this weighed material at 500 to 600°C. permits a distinction by weight to be made between the amount of organic and inorganic material in a given sample. The loss in weight is organic matter that has been volatilized or burned. The significance of suspended dissolved and settleable solids in a given sample of waste are known to all of you. Coarse and floating solids, which are easily determined by visual observation, should never be permitted to be discharged to any sewer system as they are easily separated, and may cause obstruction of the flow in the sewers and damage pumps or other equipment at the treatment works.

TABLE II

Effect on Concrete of Buffered Solutions of Acids and Bases

Duration of Test	pH of Solution	Initial Wt. of Concrete Block	Final Wt. of Concrete Block	Change in Wt. per gram	Percent Change
2[illegible] days	3	38.197 gm	24.316 gm	-13.881	-36.35
" "	4	43.504 gm	28.863 gm	-14.641	-33.66
" "	5	42.076 gm	36.087 gm	- 5.889	-14.24
" "	6	43.921 gm	42.922 gm	- 0.999	- 2.28
" "	7	50.475 gm	50.703 gm	+ 0.228	+ 0.45
112 "	8	42.326 gm	42.291 gm	- 0.035	- 0.083
11 "	9	43.752 gm	44.075 gm	+ 0.323	+ 0.74
11 "	10	39.819 gm	39.656 gm	- 0.163	- 0.409
111 "	11	39.876 gm	40.070 gm	+ 0.194	+ 0.49

as applied to industrial wastes the following comments are applicable. Colour in industrial wastes discharged to public sewers is not an objectionable characteristic so long as the dilution in the sewers and the treatment processes employed can adequately remove it before the final effluent reaches the stream. Odours are objectionable when they cause atmospheric pollution in any part of the sewerage system, and can lead to increased plant operating costs when the application of chlorine or other chemicals is used to control them.

(d) Radioactivity

Radioactive wastes like synthetic detergents are a technological development of the past two decades. At present only in a few instances are low-level radioactive wastes been discharged to public sewers on a regular basis, and while they appear to create no specific operating problems I will try briefly to relate the fate of such materials in biological processes and point out the hazard.

Due to the elaborate precautions at the hospitals or industries using radioactive material one should only find waste concentrations down in the parts per billion in sewers. These long-life radioactive inorganic isotopes are of no benefit to the biological floc but experimental evidence shows that they are absorbed onto and held by the floc. It is the resultant sludge therefore that requires special handling and separate disposal as the radioactive isotopes are held in it and will decay according to their respective half-life.

Sewage treatment processes cannot and will not be recommended as acceptable treatment methods for radioactive

wastes but in the future they may prove necessary for the reduction of BOD in the radioactive wastes themselves, and hence the treatment hazards should be understood by the operator. In the cities where the potential hazard of a careless or accidental discharge to the sewerage system exists sewer monitoring programmes have been introduced which would give adequate warning to the operator of the plant and permit him to make adjustments in treatment methods for any incoming potentially dangerous waste.

(e) Synthetic Detergent Content

Synthetic detergents are something we are just going to have to live with until the soap industry adopts or is forced to adopt substitutes which are not so resistent to biological attack. They appear not to adversely affect sewage treatment processes but their nuisance value alone (foaming problems and increased algae growth in receiving waters) are strong arguments for keeping them to a minimum in any sewerage system.

(f) Dissolved Substances

With dissolved substances particularly sulphates and phosphorous two problems arise which little affect the processes of sewage treatment . One involves crown corrosion of sewers where sewage temperatures are high and sulphate concentrations appreciable. This difficulty is associated with the generation of hydrogen sulphide gas which in turn is oxidized to sulphuric acid by bacteria which remains on the sewer walls from times of high flow.

The second involves the failure of sewage processes to utilize all the phosphorous now found in sewage flows, and this excess when discharged in effluents from treatment plants can create unnatural conditions of overenrichment in lakes and rivers leading to nuisance algae blooms. The Oakville area on Lake Ontario is an example of where this has occurred.

BIOLOGICAL CRITERIA

With industrial wastes going to municipal sewers the two most important characteristics to investigate are the oxygen consuming nature of the wastes and their toxicity. The toxicity factor is commonly considered to be characteristic of industrial effluents only.

(a) Dissolved Oxygen

The dissolved oxygen test in relation to sewage before and during treatment plays a very minor role except perhaps in

relation to research projects or to measure the efficiency of oxygen transfer in the aeration tanks. However, the dissolved oxygen is probably the most important single criterion for the measurement of pollution of natural streams by organic wastes including municipal sewage. In this regard at least 1 to 2 ppm of dissolved oxygen or preferably more should be present at the effluent end of a biological treatment process in order not to impose too much of a load on the receiving waters.

(b) Biochemical Oxygen Demand (BOD)

The 5-day figure at 20°C. with which you are all familiar is generally used, and in that time only part of the oxidizable matter is decomposed. The carbonaceous matter is preferentially oxidixed with the oxidation of nitrites to nitrates taking place at a later stage. Under the 5-day restriction for a readily oxidized organic waste about 68 per cent of the ultimate oxygen demand is realized.

Certain industrial wastes are known to contain compounds resistant to biological breakdown while others may be amenable to oxidation but the organisms present in the sewage are unsuitable or require acclimatization. Compounds which interfere with this test are listed in Table III.

TABLE III

Effect of Various Substances on the Five-day BOD

Substance	Concentration ppm	Reduction of BOD Result (approx.)
Free Chlorine	0.005	5%
Chromium Trivalent	1.0	10%
Hexavalent	0.45	10%
	4.0	70%
Cobalt Chloride	0.9	5%
Copper	0.4	92%
Cyanide	0.1	5%
	1.0	40%
	25.0	100%
Lead	0.2	5%
Mercuric Chloride	9.025	20%
	2.0	100%

(g) Oils and Greases

Oils and greases in wastes fall into two classes: saponifiable and non-saponifiable. The sponifiable are what are known as glycerides of fatty acids (the basis of soaps, etc.) and are subject to both aerobic and anaerobic decomposition. The non-saponifiable are principally of mineral origin and are not easily decomposed under any conditions. Such oils could reach the sewers from garage and filling stations or through the inadvertent escape of fuel oil at an industry. The discharge of all such materials should be prohibited by ordinance, although in many towns control leaves much to be desired.

CHEMICAL CRITERIA

The physical criteria of pollution depends largely on foreign substances that are not dissolved in the aqueous. Chemical pollution on the other hand is usually caused by dissolved materials that are completely dispersed throughout the water phase in molecular or ionic form, and are therefore not removable by mechanical operations like settling and filtering.

(a) pH, Acidity and Alkalinity

In sewage treatment employing biological processes, pH must be controlled within a range favourable to the particular organisms involved. For this reason, and to prevent sewer corrosion pH must be controlled at the discharge point to the sewer. Sewage arriving at a plant below pH 6 or above pH 9 can be suspected of containing an industrial waste which is not receiving adequate pretreatment before discharge.

Acidity and alkalinity are measured by determining the quantity of alkali or acid, respectively, needed to restore the solution to pH 7, neutrality, or some other specified pH value. They are often said to be a measure of the degree of difficulty one would encounter in neutralizing a waste and are usually expressed in parts per million of calcium carbonate.

With such criteria as

(b) colour and
(c) taste and odour

(c) Other Oxygen Consumed Tests

Two other tests often used in conjunction with the 5-day BOD test or to supplement it involve the oxidizing of organic wastes chemically with permanganate or dichromate solutions and measuring the amount of oxidizing reagent consumed.

In many industrial wastes a relatively constant relation between the oxygen consumed tests and the BOD has been established and these tests then become a useful adjunct to the slower BOD determination. Chemical methods are particularly useful in the analysis of industrial wastes containing toxic substances that inhibit biological growth and render the BOD test unreliable.

(d) Toxicity

The most troublesome discharges of toxic inorganic and metallic ions originate from the plating and metal-finishing industries whose wastes contain variable concentrations of cyanide, chromium, nickel, copper and zinc. The problem in regard to control to protect secondary treatment processes arises in calculating from a discharge at the industry the concentration which will reach this stage of the process, because chemical change occurs as well as possible precipitation and occlusion in sludge in the primary system. Other factors to consider are the degree of acclimatization of the biological process, and the degree of dissociation of the compound in solution.

In the intermediate course Mr. Millest covered in some detail the specific metallic ion concentrations which affect biological oxidation processes and they will not be repeated at this time. Very effective pretreatment methods have been developed to handle these wastes at their source and many industries in Ontario especially the larger ones have installed this equipment.

(e) Nitrogen Compounds

The proper amount of organic nitrogen is essential for the successful operation of biological oxidation processes as it is one of the most important nutrients or food materials on which micro-organisms feed. Besides organic nitrogen there are other forms such as ammonia nitrogen, nitrites and nitrates which play a role in sewage treatment.

In biological treatment processes, organic and ammonia nitrogen are first converted to nitrites by oxygen and by taking up additional oxygen are converted to nitrates. If the treatment works does not fully accomplish this and few do, the

river or lake into which the effluent is discharged will ultimately do so, often leading to serious algal growths. On a sewerage system the presence of large amounts nitrogen deficient wastes could seriously affect treatment at the plant. Cotton kiering waste, brewery waste and rag-rope pulping wastes are examples.

(f) Direct Uptake of Oxygen

Limited use has been made of a device known as a Warburg manometer to get a direct measure of oxygen uptake on certain wastes because of the questionable value of the BOD test for measuring the biological oxidizability of industrial wastes. With this method it is possible to ascertain whether substances in the waste will prevent the proper functioning of bacteria and it could possibly be used to determine the optimum conditions for industrial waste sewage mixtures in an activated sludge plant.

TYPES OF AERATION DEVICES

IN SEWAGE TREATMENT PLANTS

by

B. Porter

Operations Engineer - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
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TYPES OF AERATION DEVICES IN SEWAGE TREATMENT PLANTS

by

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The most important, the most costly, and often the most troublesome factor in the activated sludge process is the introduction of air into the aeration tanks. This has spurred the ingenuity of engineers since the inception of the process, and in particular to devise better methods of aeration.

The ultimate aim of any aeration system is to supply the oxygen requirements of the living organisms in order that they may, in their life process, purify the incoming waste flow.

This lecture will present some of the more common devices used to accomplish this purpose.

AIR DIFFUSION DEVICES

(1) Ceramic Plates and Tubes

Ceramic plates and tubes have been in use for many years and have proved to be one of the more practical methods of aerating sewage. Fused quartz plates known as FILTROS plates and fused alumina sold under the trade names of Aloxite, Norton and Alundum are quite common. A few are tabulated below.

Manufacturer	Material	Standard Dimensions (in.)		
		Plates	Tubes	Range of Permeability Available

The Carborundum Co.	Grains of crystalline Aluminum oxide bonded with high alumina glass	12x12x1	24x3IDx5/8 24x2IDx1/2 24x13/4IDx3/8 up to 120
Filtros Inc.	Natural pure silica sand bonded with synthetic silica	12x12x1 1/2	up to 80
Norton Co.	Grains of cyrstalline aluminium oxide bonded with high alumina glass	12x12x1	24x3IDx5/8 24x2IDx1/2 24x1 3/4Idc/8 up to 120

(2) "Precision" Tubes

"Precision" Diffuser tubes were introduced by Chicago Pump Co. in 1947. The core is formed of a welded longitudinally corrugated stainless steel cylinder, 3in. I.D. and 24 in. long. The core is wrapped with three strand twisted Saran cord. Saran is a co-polymer vinylindene chloride plastic which is claimed to be non-water absorbent, and resistent to acids, alkalies, organic and inorganic solvents. The tube weighs approximately 3 1/2 pounds. The nominal capacity of each tube is 6 cfm. The head loss varies from 6 to 12 in. of water at 2 to 7 cfm. respectively when the inlet control orifice is used.

(3) "Colaflex" Diffuser

The "Colaflex" Diffuser was introduced by Infilco Inc. in 1951. The diffusing element consists of a synthetic fabric formed in the shape of a hood fastened to a pan made of fiberglass reinforced plastic. The fabric is fastened to the pan using a gasket and stainless steel band. The fabric is made of woven seamless material and is flexible and collapsible. Air under pressure inflates the element to a distended position when diffusing through the fabric to the liquid. The nominal capacity of each unit is 6 cfm but air flows of 1 to 10 cfm can be handled. The head loss across the fabric varies from 6 to 12 inches of water at nominal flow. It is claimed that collapsing of the fabric when air pressure is released cleans the diffuser.

(4) "Platetube" Porous Diffuser

Walker Process have recently introduced their "Platetube" diffuser which consists of a ceramic flat plate manufactured in 12

and 24 in. lengths, four and six in. widths, and 1/2 in. thick. The porous media is aluminium oxide and is manufactured under the name Aloxite. The Aloxite used in the Platetube diffuser has a permeability of 40. It is recommended that filtering the air supply to a concentration of particles of not greater than 0.1 mg/1000 cu. ft. to avoid permanent input clogging. This diffuser combines the operating characteristics of the older porous plate diffusers with a retractable header assembly.

(5) "Flexofuser" Tubes

These tubes are manufactured by Chicago Pump Co. and consist of saran cloth attached over a fiberglass body 24 in. in length and 1 1/2 in. in diameter. These tubes may be used under standard conditions, but are especially recommended where external clogging is anticipated or experienced. Air supply should be filtered to a concentration of particles of not greater than 0.1 mg/1000 cu. ft. It can be collapsed similar to the colaflex diffuser to clean in place.

(6) Jet (Impingement) Aeration

Impingement aeration equipment consists of two manifolds placed in the aeration tank; an air header which is on the underside and parallel to a recirulated tank liquor header. These units are made up in headers up to 40 ft. long, with impinger bowls and water nozzles saddle mounted to the air and water headers at from 15 to 24 in. centres. In this type of diffusion, a high-velocity jet of water is combined with air bubbles released within an open cuplike impinger bowl. The cavitiation effect produced by the impinging jet of water divides the air into extremely fine bubbles. The impingement water is pumped by built-air lift pumps, recycling the aeration tank liquor. Bubble size can be controlled by the amount of impingement liquor delivered by the circulating air-lift pump.

The impinger bowls normally have four 3/8 in. air outlet holes and can diffuse from 4 to 16 cfm of air each. The impingement water nozzle is tapered, making itself cleaning when the direction of water flow is reversed by periodically reversing the air-lift pump. The outlet of the water nozzle is normally 5/8 in. in diameter and requires 15 to 18 gpm of impingement liquor. The power required for pumping impingement water is from 8 to 15 percent of the total power for aeration. It is claimed that this type of diffusion has a low head loss and requires no air filters.

(7) Sparger Units

Spargers consist of a cast iron body equipped with multiple nozzles of varying diameters, with ports drilled, reamed and faced.

The spargers are saddle mounted or screwed into the header and are uniformly spaced on 15 to 24 in. centres. Each sparger releases air through four or eight high velocity horizontal orifices. The spargers can diffuse from 4 to 18 cfm of air each. It is claimed that the head loss is low and no air filter is required.

(8) Discfuser Unit

Discfusers are a cast iron body equipped with a 3 in. diameter floating plate secured by a stainless steel retainer ring. When in operation the circular plate is in the floating position, thus permitting air to discharge through a peripheral slot. If the air supply to the units is shut down the disc drops, preventing liquor from entering the air header. Discfuser units are comparable in performance to spargers.

(9) "Shearfuser" Diffusers

The "Shearfuser" is simply a sheet metal box 7 in. square and 12 in. deep, open at the top. The box is placed in the bottom of the aeration tank and air is discharged from an open pipe (3/4 in diam.) at the bottom centre of the box. The air forms bubbles in the water and the resulting air-water mixture being lighter than water flows up and out of the shear box into the aeration tank. Water flows into the box to replace that which is carried out in the air-water mixture. The centrally rising air-water mixture and the downward moving replacement water all flowing within the limited space of the shear box create a turbulence or hydraulic shear which breaks the large bubbles emanating from the air inlet pipe into many smaller bubbles and develops the localized turbulence desired for increasing oxygenation efficiency.

(10) Venturi-Type Diffuser

The venturi-type diffuser was developed to utilize hydraulic shearing forces. It is a short venturi tube open at both ends, placed in a vertical position in the aeration tank. In operation, air is discharged from a pipe at or below the throat of the venturi and the air-water mixture flows up through the venturi and dicharges into the tank. The difference in the specific gravity of the air-water mixture in the venturi and its discharge pipe as compared to the specific gravity of the water surrounding the venturi provides the necessary head for creating the flow through the venturi, as in any air lift pump. However in the venturi diffuser the energy is all expended in accelerating the water through the venturi since there is no static lift. At the venturi throat, the shearing forces within the low break the large bubbles issuing from the air pipe into many small bubbles.

(11) Efficiency of Air Diffusion Devices

Many investigations have been performed in the past few

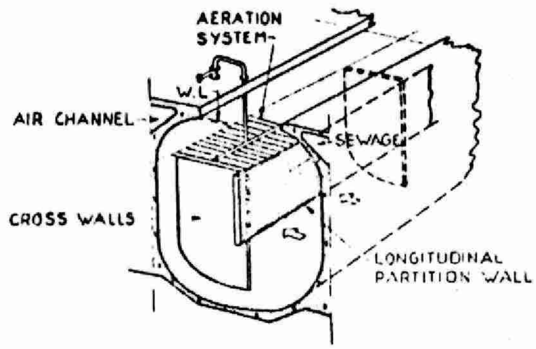


Figure 1a - Low Head Aerator Design

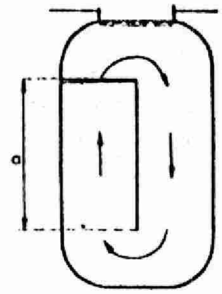


Figure 1b - Straight Parts of Circulation Channel

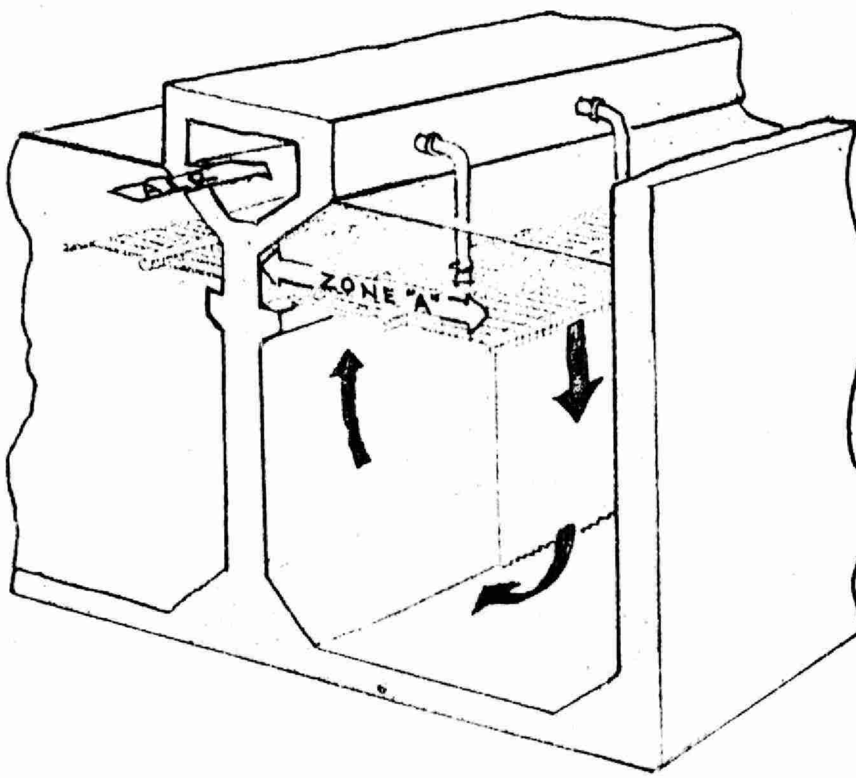


Figure 1c - Cross-section drawing through INKA channel

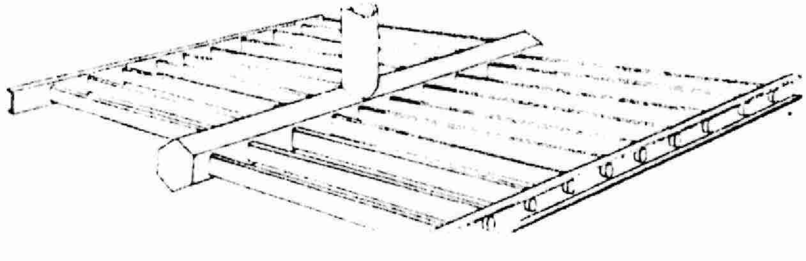


Figure 1d - Aeration Grid

years to determine the efficiency of various air diffusion devices. Most of the investigations are conducted in the Laboratory using tap water and many with a dissolved oxygen content of zero. The term efficiency in this case is used to indicate the quantity of oxygen absorbed by the liquid in per cent of that supplied to the diffuser. It is to be remembered that efficiencies which are based on tap water and with a dissolved oxygen content of zero are relative only. Actual transfer efficiency into a biological suspension with 2ppm dissolved oxygen will be lower. In general the actual efficiency in a treatment tank will be 60 to 65% of that given for tap water and zero dissolved oxygen.

The following are the efficiencies reported by various investigators using tap water, zero dissolved oxygen, a diffuser submergence of 12 to 13 feet and rated air capacities, -

"Precision" Tubes	-10-12%
Spargers	- 6-8 %
Discfusers	- 6-8 %
Shearfusers	- 7-9 %
Venturi Type	- 7-9 %

LOW PRESSURE AERATION

At unchanged power consumption it is, of course, possible to use more air at a reduced aerator depth than at greater depth. In a system with no friction losses the rate of air at the same input of energy is inversely proportional to the depth. Thus we can use from 7 to 10 time as much air at 1.64 ft. depth as at a normal depth. It has been stated that the absorption rate is very great at formation of the bubbles; however, during a fraction of the initial second it decreases considerably to a low but fairly constant value. It therefore may be worthwhile to investigate the possibilities of producing new air-water interfaces with large amounts of air at shallow depth.

BRIEF DESCRIPTION OF THE D.O. INKA AERATION SYSTEM

Aeration Tank

As illustrated in Figure 1(a) and (c) the aeration tank is partially divided into two compartments by a vertical baffle, beginning at the point of air admission and extending downward to a point above the tank floor. Air is admitted into the upper zone "A", Figure 1(c) through an aeration grid consisting of a header and cross pipes extending at right angles to the header. The introduction of large volumes of air in such a manner creates an air lift effect causing mass circulation upwards in one section and downwards in the other.

Source of Air

In this system air is supplied by centrifugal fans, not unlike those found in air-conditioning practice. Since the aeration grids are located at a relatively shallow depth below the tank liquid level, the total head on the head fan system is generally about 35 in. water pressure (1.25 psig.) The need for air filtering equipment is eliminated. Only screens or louvered gratings are provided to keep out leaves, sticks etc.

Air Piping

Air system piping is larger in diameter than that of conventional systems, however, thin wall light-weight ducting is used since pressurers are low. The air to be supplied via each individual aeration grid can be transported to the tanks through head-channels formed by the "Y" section of the tank coping at the top of each middle wall. If desired, the fan can be placed at one end of the tank directly adjacent to it and either supplied with weather-proof fixtures or housed in a simple enclosure. This practically eliminates air piping.

Service Pipe and Aeration Grid

The aeration grid (Figure 1(d) consists of a common manifold with a series of tubes extending at right angles to the manifold. The orifices are located at an angle from the vertical on the underneath portion of the tube and alternate from side to side. The ends of the tubes are equipped with removable caps for inspection of the inside walls of the tubes. One of the grids and service pipe weighs approximately 50 pounds, and may be removed for service.

Tank Baffle

The baffle support is fabricated of steel members forming a lightweight truss extending from end to end of the aeration tank. The baffle itself is made of corrugated fiberglass. The baffle sections are anchored at each end of the aeration tank and periodic tie-ins are made with the side walls for lateral stability.

MECHANICAL DEVICES

Basic Considerations

Basically, in all surface aerators, there is created a region of intense turbulence and air-water interfaces at the surface of the treatment tank through which the oxygen deficient liquid is pumped. The proper balancing of (1) the energy put into pumping and circulating the liquid, and (2) the energy used for oxygen input, ie., creating air-water interfaces determines the efficiency of a particular unit.

In order to compare various oxygenating schemes and equipment, it is necessary to express quantitatively the two functions previously mentioned under standard conditions. Oxygen input should be expressed in terms of pounds of oxygen per unit time per unit of power; normally given as lbs. O_2/hr/HP or lbs. O_2/KWH.

So far as mixing characteristics are concerned the most significant designation is the liquid pumpage or circulation rate. A given amount of power put into, for example, a radial flow turbine, will produce different liquid pumpages, depending on the velocity of the issuing liquid. It can be demonstrated that for a given power input a larger quantity of liquid can be moved if the movement occurs at lower velocities. However this fundamental idea must be considered in connection with the minimum velocities that are necessary in order to keep solids in suspension. High speed turbines can induce intense turbulence and good air entrainment; however, a large amount of kinetic energy put into the liquid will result in reduced flow. In other words, adequate mixing cannot necessarily be obtained with high speed mechanical pumping devices, even though they may create localized regions of intense turbulence.

The design of a mechanical surface aerator is not merely the building of a device that will produce intense splashing and localized turbulence. Considerable hydraulic design work, testing and field verification under full-scale conditions is required before an optimum surface aerator design can be made in regard to best oxygen input efficiency.

DRAFT TUBE TYPES

"Simplex High Intensity" Aerator-Ames Crosta Mills and Co. Ltd.

The Simplex Surface Aeration process was originally developed in the early 1920's. In the late 1940's new type of aerating cone, known as the Simplex High Intensity Aerating Cone was developed. See Figures 2 and 3. The fundamental object of the process is to keep the mixture of sewage and activated sludge

in continuous circulation within the treatment tank, and to provide intensive aeration by spraying the mixed liquor over the surface of the tank. This is achieved by an uptake tube and aerating cone. The cone is in effect a low lift pump, which lifts the mixed liquor a few inches above general water level. The blades are specially designed to spread the liquor across the water surface.

The cone is positioned in the middle of the aeration channel at water level. Below the cones is a stationary uptake tube which terminates at the bottom of the channel in a bellmouth supported several inches above the tank floor by three adjustable feet. A seal is formed by the spigot end of the uptake tube entering an anular airlock chamber in the inlet skirt of the revolving cone. Above the inlet shirt, the "High Intensity Cone" comprises an open-ended conical shell formed from steel plat, to the inner face of which are attached a number of specially shaped blades. The upper lip of the conical shell is arranged several inches above the water level.

Normal operating speeds of the cone range between 30 and 50 rpm. The rate of circulation provided by the cone is sufficient to achieve a complete turnover of liquid approximately every five minutes.

There are two methods of varying the oxygen transfer rate. For daily variation a two sided weir positioned at the outlet of each aeration channel may be adjusted over a range of approximately six inches, thereby varying the intensity of aeration. A chain drive is used between the motor and lineshaft which allows the rate of rotation of the cones to be changing the driving sprocket.

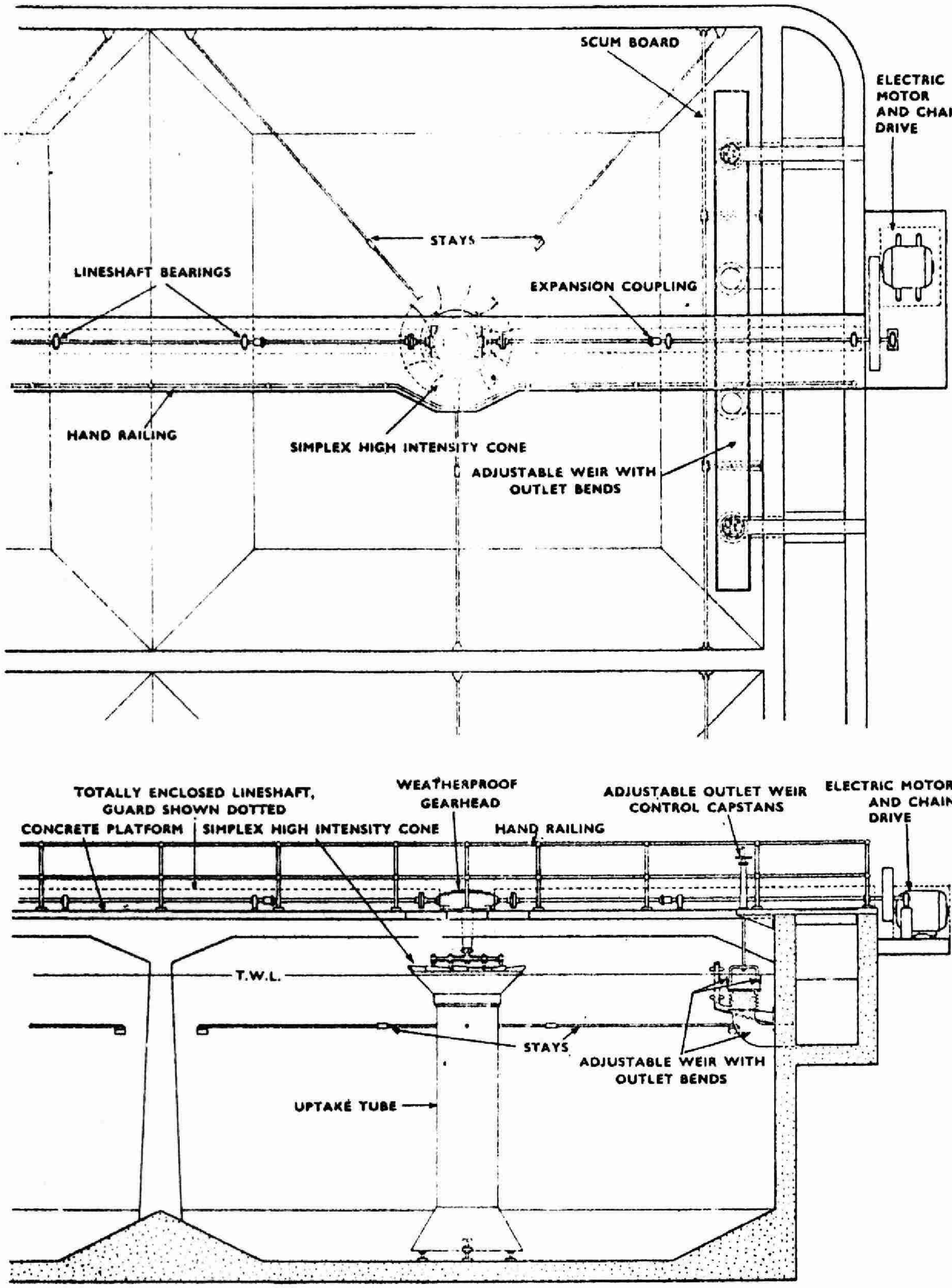
In large plants each cone serves an area of 900 to 1100 sq. ft. and an aeration tank volume of 60,000 to 85,000 gals. The largest installation of this type treats an average dry weather flow of 96 MGD and comprises 384 cones.

Field Tests

Tests conducted on the "Simplex" aerator at the Sandford Sewage Disposal Works, England (average daily flow 5.5 MGD), by the Water Pollution Research Board.

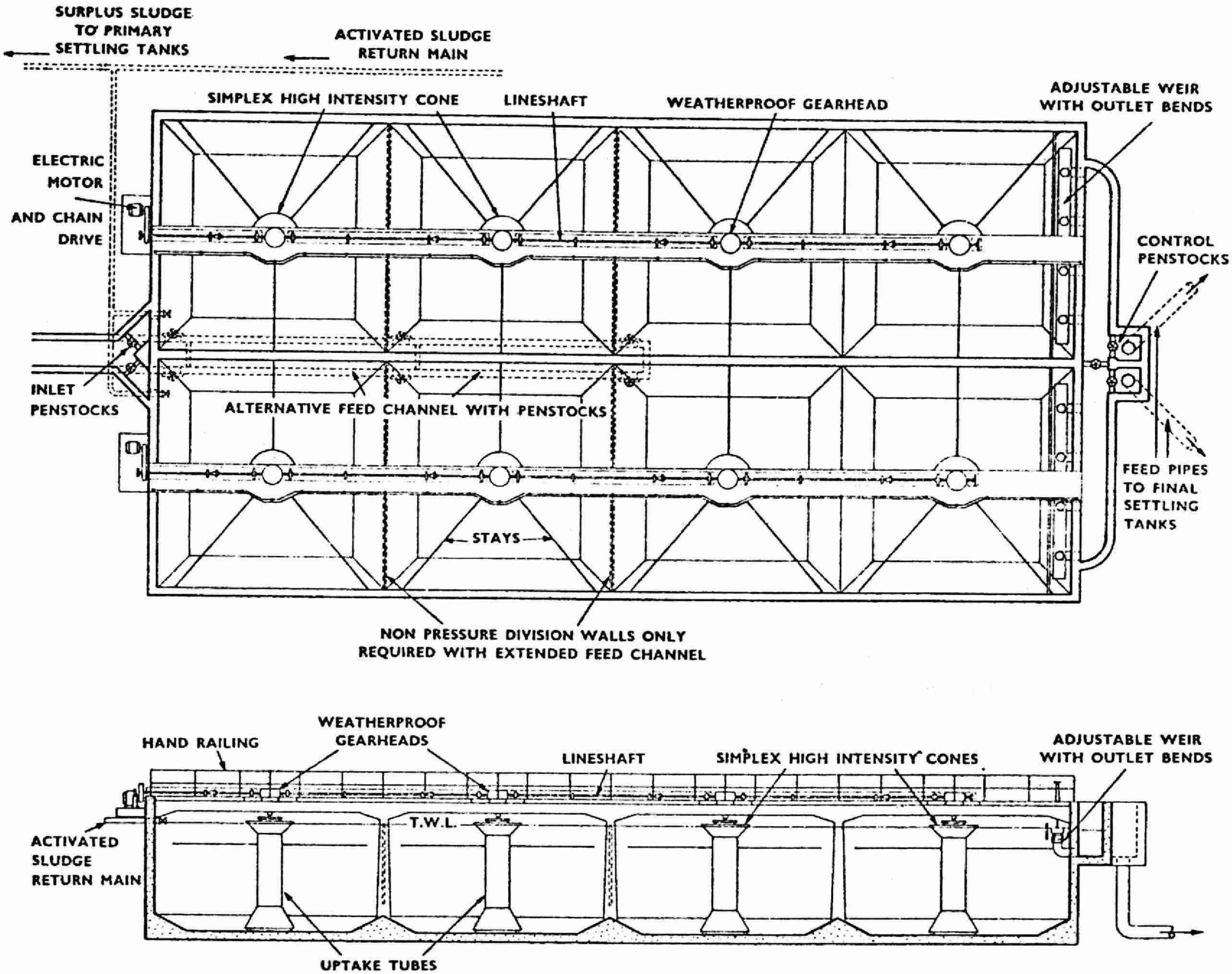
In Figure four it can be seen that increasing the rate of rotation of the cones from 26.4 to 37.1 RPM increased the oxygenation capacity roughly 2.5 times, ie., oxygenation capacity varies approximately as $(RPM)^{2.5}$, an effect similar to that found with some other types of mechanical aerators.

The effect of varying the freeboard (distance from lip of cone to still water level) appears to be relatively small over the range investigated.



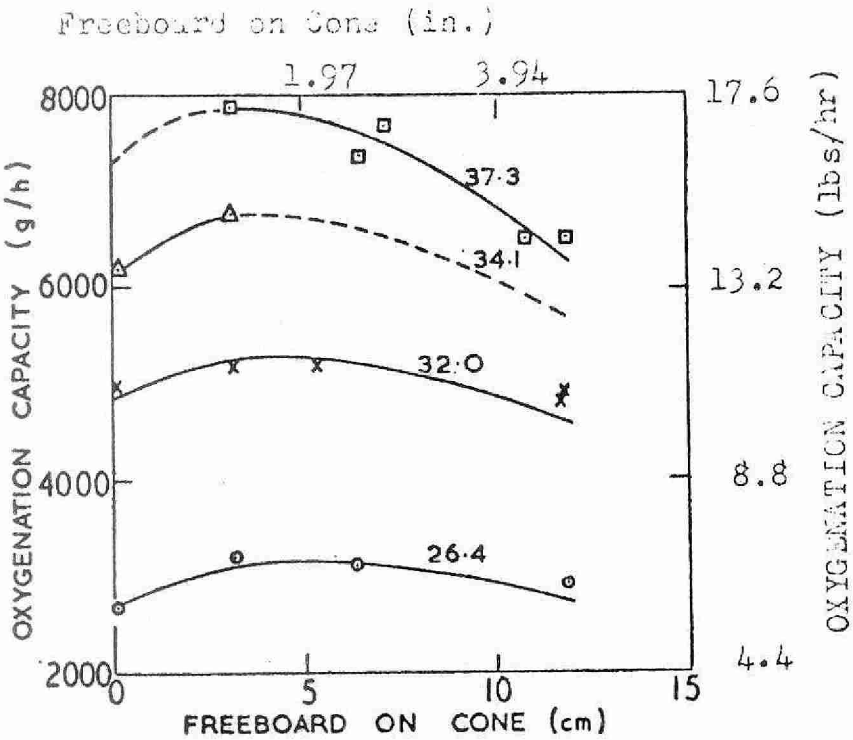
Equipment of a typical Simplex High Intensity Surface Aeration Plant

FIGURE 2



Typical layout of a Simplex Surface Aeration Plant with 8 High Intensity Cones.

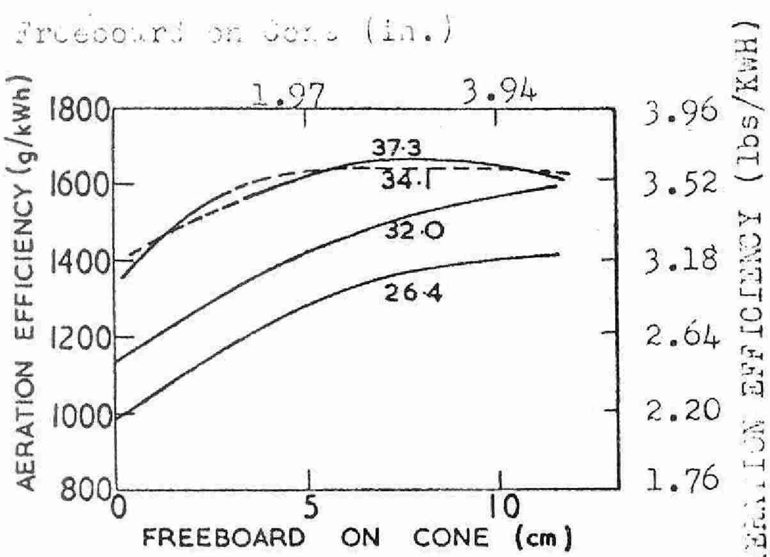
FIGURE 3



Oxygenation capacities of "Simplex" aeration units at Oxford sewage works

Rate of rotation of cone in rev/min shown against each curve. Broken lines indicate interpolation or extrapolation

FIG. 4



Aeration efficiencies of "Simplex" aeration units at Oxford sewage works

Rate of rotation of cone in rev/min shown against each curve. Broken lines indicate interpolation or extrapolation

FIG. 5

Figure five gives the aeration efficiency in terms of net output power of the motors. It appears that at low freeboards the efficiencies are higher at the speeds but with increasing freeboard the efficiencies at the lower speeds approach those for greater speeds. The highest value observed, 3.74 lbs/KWH net, is comparable within the range obtained with several other mechanical aerators.

"HI-CONE" MECHANICAL AERATOR - YEOMANS BROTHERS CO.

The "Hi-Cone" aerator is quite similar to the "Simplex" unit except for minor modifications of the impeller design. Tank installation and operation are the same as for the "Simplex" aerators.

Field Tests

Field tests were conducted at the Glendale Sewage Treatment Plant by the Metropolitan Sanitary District of Greater Chicago.

It can be seen from the graphs that within the limits of the tests, oxygen absorption increases as the speed of the cone increased.

The effect of varying the freeboard (Figure 6) in the 28 to 39 RPM range has a relatively minor effect on the efficiency. It can also be seen in Figure six that at low freeboards, variations in speed have a smaller effect on efficiency that at higher freeboards, which is the reverse of that shown for the Simplex aerator.

The best efficiency obtained (4.4 lbs/KWH net), for this investigation, appears to be slightly higher that other aerators of this type.

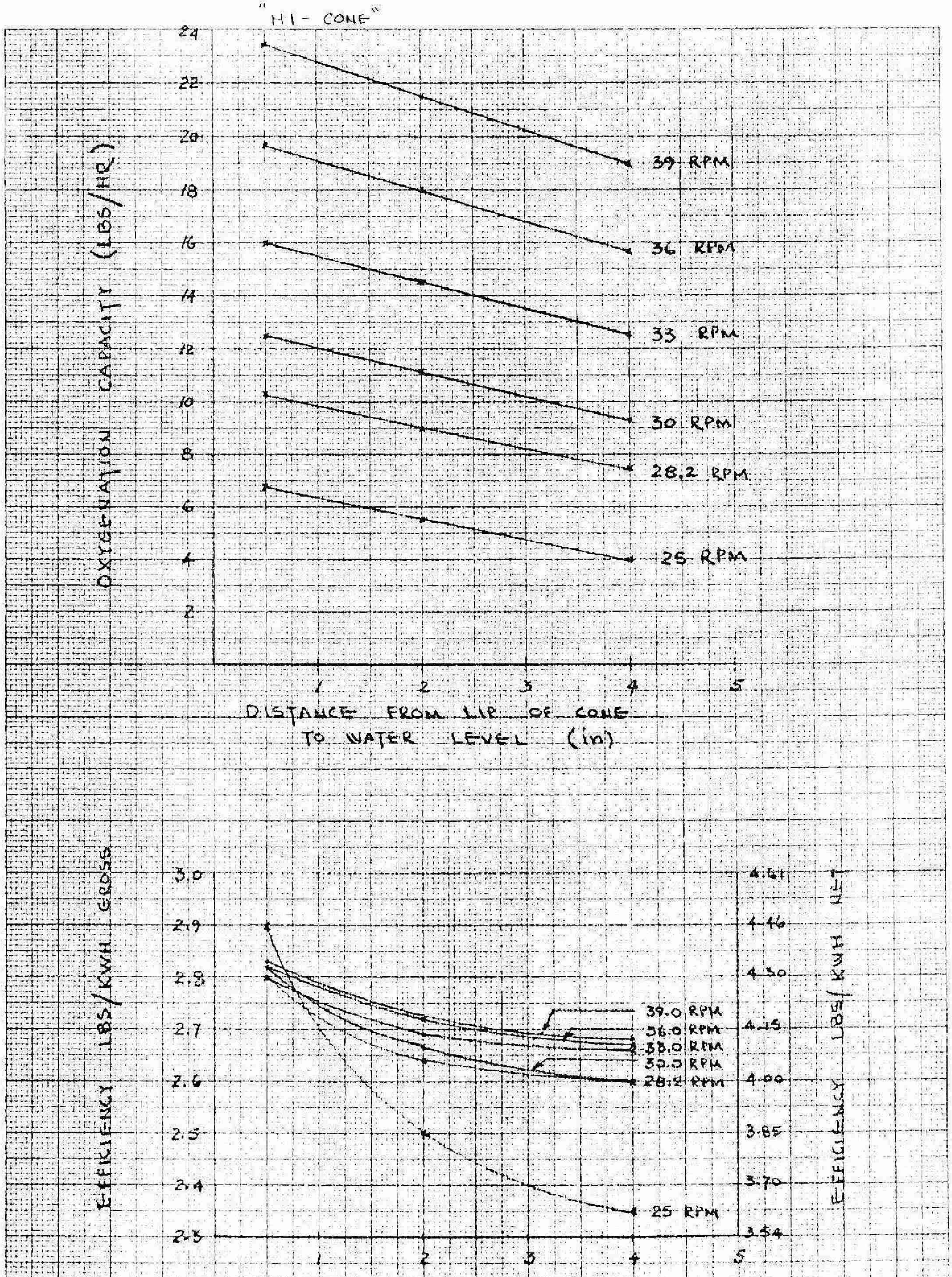
TYPE "S" MECHANICAL AERATOR - CHICAGO PUMP CO.

The fundamental object of this aerator is the same as the "Simplex" system ie., to ensure conditions circulation of tank contents and to provide aeration by spraying the mixed liquor over the tank surface. In this case this is accomplished by an uptake tube, a propellor and a deflector cone. See Figure seven.

Field Tests

Oxygen transfer tests were performed on the Chicago Pump Co. type "S" mechanical aerator at the Huntsville, Ontario Sewage Treatment Plant.

The efficiencies of aeration ranged between 0.46 and


"HI- CONE"
24
22
20
18
16
14
12
10
8
6
4
2
OXYGENATION CAPACITY (LBS/HR)
39 RPM
36 RPM
33 RPM
30 RPM
28.2 RPM
25 RPM
1
2
3
4
5
DISTANCE FROM LIP OF CONE
TO WATER LEVEL (in)
3.0
2.9
2.8
2.7
2.6
2.5
2.4
2.3
EFFICIENCY LBS/KWH GROSS
4.61
4.46
4.30
4.15
4.00
3.85
3.70
3.54
EFFICIENCY LBS/KWH NET
39.0 RPM
36.0 RPM
33.0 RPM
30.0 RPM
28.2 RPM
25 RPM
1
2
3
4
5
DISTANCE FROM LIP OF CONE
TO WATER LEVEL (in)

184 FIGURE 6

0.50 KWH/lb of oxygen dissolved. (2.17 and 2.0 lbs. O_2/KWH). In terms of power consumed per lb. of oxygen absorbed this is slightly higher than for other mechanical aerators. Their calculations are, however, based on gross power consumption, ie., the power consumed by the motor; motor efficiency was not considered.

NON DRAFT TUBE TYPES

"Vortair" Aerator

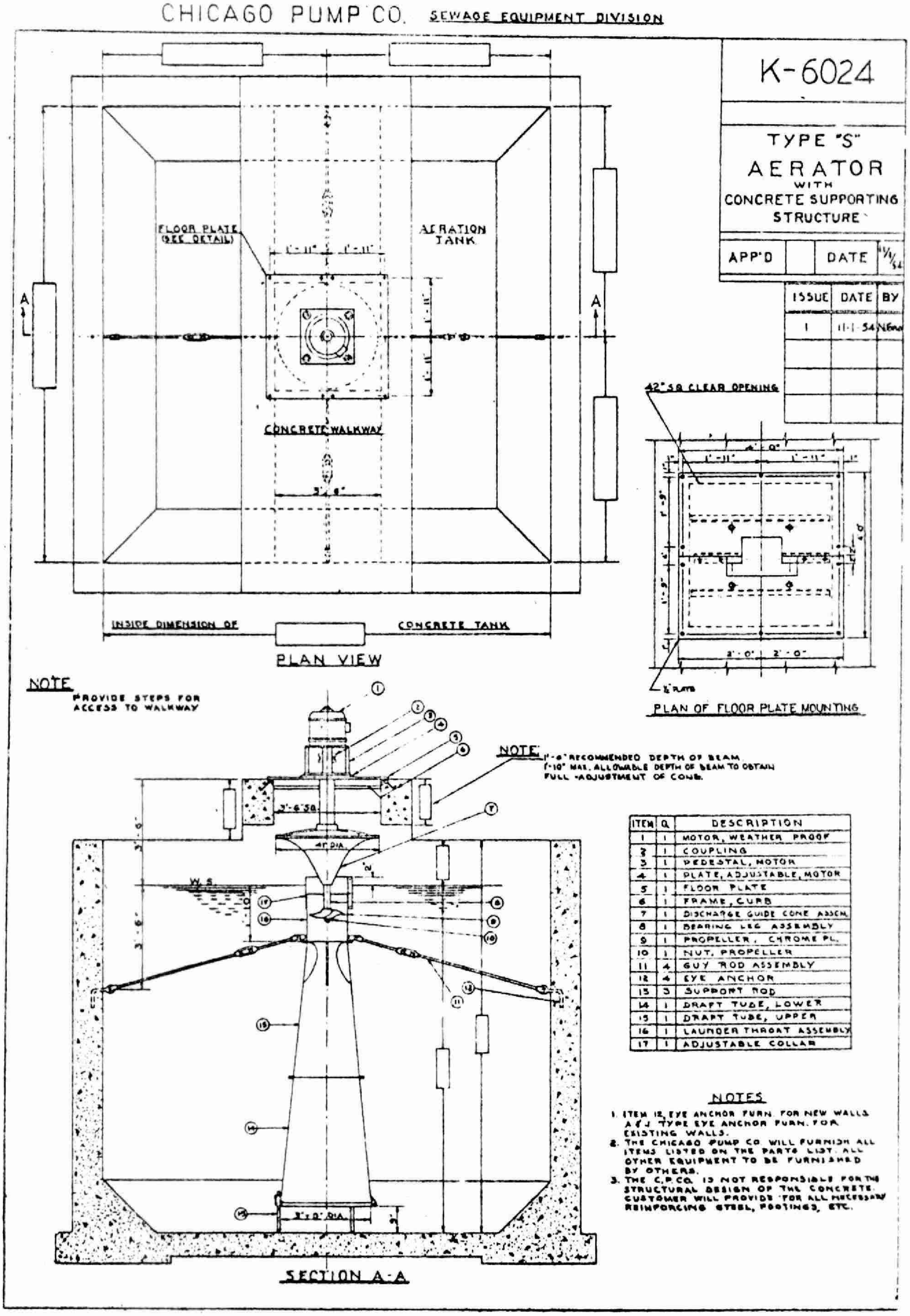
This Aerator (Figure 8) includes a motor-reducer drive, a shaft, and a horizontally-mounted circular flate plate from which a series of radial blades extend downward. Rotor blades are equally spaced and have their outer edge flush with the periphery of the plate. The entire weight of the shaft and rotor is carried by the drive unit and no submerged bearings are used.

When such a turbine is submerged a specific distance below the liquid surface, which distance depends on the turbing diameter and its peripheral speed, the pumpage of liquid from the turbine is at such a velocity as to produce a "jump". To operate in this manner, submergence of the rotor is reasonably critical. If the turbine is submerged excessively, the power increases and a well defined jump is not produced. When operating under optimum conditions, the top plate of the turbine is completely free of liquid, even though under quiescent conditions it is submerged several inches. Also, if the turbine is located too close to the liquid surface, a well defined jump is not produced, but rather a spraying action, which produces good oxygen input, so far as power consumption is concerned, but the pumping action it too weak.

Photographic studies revealed the fact that the liquid issuing radially from the turbine already had fine air bubbles entrained in it. It was discovered that there was a pocket of air on the downstream side of each of the blades. High velocity waters streams issuing radially between the blades were causing a shearing action which resulted in fine bubble entrainment as the liquid left the turbine.

Behind each blade a region of reduced pressure was created, and with the proper location of the top of the turbine with respect to the liquid surface, it was possible for atmospheric air to be drawn down past the edge of the turbine plate into the low pressure areas. This observation led to a modification wherein holes were drilled in the rotor plate on the downstream side of each of the radial blades. The vented plate improved the oxygenation efficiency and also steadied the flow from the turbine.

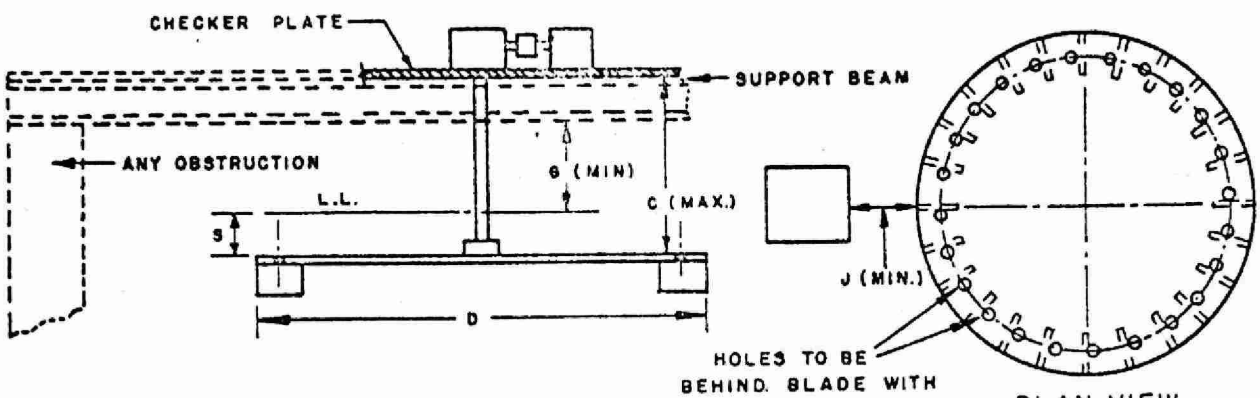
Tests were also made in regard to the circulation velocities. At first, it was thought necessary to install a



ITEM	Q.	DESCRIPTION
1	1	MOTOR, WEATHER PROOF
2	1	COUPLING
3	1	PEDESTAL, MOTOR
4	1	PLATE, ADJUSTABLE, MOTOR
5	1	FLOOR PLATE
6	1	FRAME, CURB
7	1	DISCHARGE GUIDE CONE ASSEM.
8	1	BEARING LEG ASSEMBLY
9	1	PROPELLER, CHROME PL.
10	1	NUT, PROPELLER
11	4	GUY ROD ASSEMBLY
12	4	EYE ANCHOR
15	3	SUPPORT ROD
14	1	DRAFT TUBE, LOWER
15	1	DRAFT TUBE, UPPER
16	1	LAUNDER THROAT ASSEMBLY
17	1	ADJUSTABLE COLLAR

 FIGURE 7

"VORTAIR" AERATOR STANDARD SIZES FIGURE 8



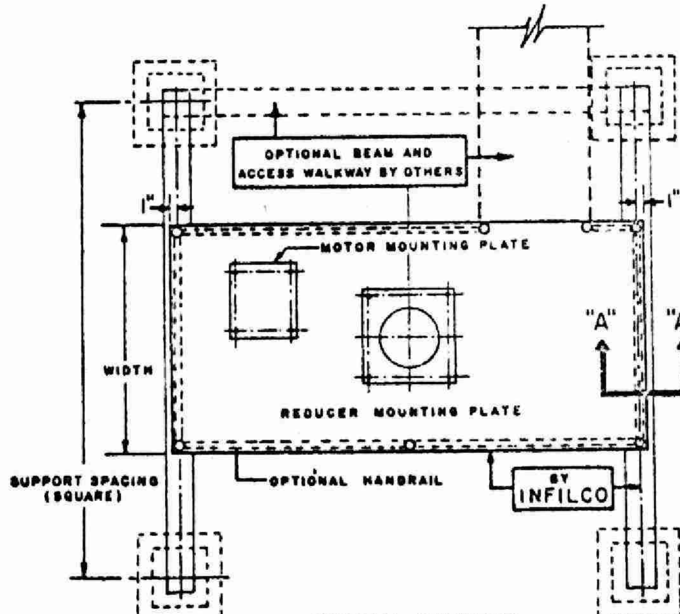
PLAN VIEW

Rotor No.	Rated[1] Capacity Lbs. O_2 P.D.	Rotor Dia. D	Torque Lbs. - Ft.	Motor HP	Weight (Lbs.)		Installation Dimensions					
					Drive	Shaft & Rotors	C Max.	G Min.	J	S Min.	S	S Max.
1	90	1'-9"	25	1	135	70	3'- 5"	1'-6"	1'-3"	2 1/8"	2 1/2"	3 1/4"
2	135	2 -0	50	1 1/2	220	75	4 - 6	2 -0	1 -6	2 3/8	2 3/4	3 5/8
3	180	2 -3	70	2	225	90	4 - 4	2 -0	1 -6	2 5/8	3	3 7/8
4	270	2 -9	135	3	220	110	4 - 3	2 -0	1 -6	3	3 1/2	4 1/2
5	450	3 -3	260	5	345	140	3 -11	2 -0	2 -0	3 3/8	4	5 1/4
6	680	3 -9	450	7 1/2	800	185	4 - 9	2 -6	2 -0	3 7/8	4 1/2	5 7/8
7	900	4 -6	700	10	890	250	4 - 6	2 -6	2 -0	4 1/4	5	6 1/2
8	1350	5 -6	1260	15	1295	445	5 - 2	3 -0	2 -6	4 5/8	5 1/2	7 1/8
9	1800	6 -3	1940	20	1495	620	7 - 6	3 -0	3 -0	5 1/8	6	7 3/4
10	2250	7 -3	2760	25	2550	950	6 - 9	3 -0	3 -0	5 1/2	6 1/2	8 1/2
11	2700	8 -3	3900	30	3250	1340	6 - 4	3 -6	3 -0	6	7	9 1/8
12	3600	9 -6	6000	40	4300	1760	8 - 7	3 -6	3 -0	6 3/8	7 1/2	9 3/4
13	4500	10 -6	8400	50	4900	2050	8 - 2	3 -6	3 -6	6 3/4	8	10 3/8
14	5400	11 -6	11000	60	6030	2400	7 -11	4 -0	3 -6	7 1/4	8 1/2	11
15	6800	13 -3	15700	75	7410	3200	9 - 9	4 -0	3 -6	7 1/2	8 3/4	11 3/8

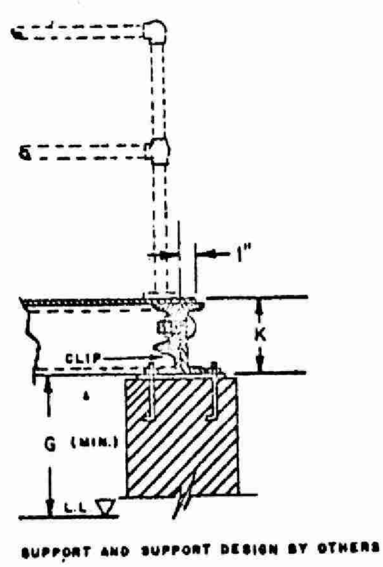
1. For liquids with oxygen absorption characteristics close to those of water, 2 ppm dissolved oxygen, and temperature not over 85° F.

2. Based on G minimum.

SUPPORT PLATFORM FOR COLUMN OR RAFT DESIGN



TYPICAL SUPPORT PLATFORM



SECTION "A-A"

draft tube below the turbine to ensure that the liquid would be pulled up from the bottom of the basin. Observations indicated that the use of such a draft tube, increased the power consumption, but had no effect whatsoever on the velocities and general circulation pattern in the basin. With the use of dyes it could be seen that the turbine was generating a vortex immediately beneath it which penetrated to the bottom of the basin. The vortex was so well defined that it could be considered as producing the exact effect desired when a draft tube is utilized without introducing the energy losses.

Thus, though the original investigations were begun with the idea of using a vortex to entrain air, the final design resulted in the use of the vortex for mixing and the use of the hydraulic jump for air entrainment.

Oxygen Transfer Tests

The total oxygen transfer is the sum of that obtained in the hydraulic jump and that obtained by direct turbine entrainment. It may be expected that the transfer per unit of power will build up to a peak and than decrease. This has been found to be the case. Oxygen transfer per horsepower increases and then decreases as the submergence is changed from lesser to greater values. Oxygen transfer rates as high as about 4.8 lbs/KWH were obtained in experiments conducted by R.F. Weston Inc., Consulting Engineers.

Kessener Brush

Another form of surface aerator that was developed in Germany and is used fairly extensively in Europe is the Kessener brush aerator. Basically, this aerator consists of a horizontally driven shaft from which extend so-called prongs or brushes made of metal. The brushes are frequently composed of acute triangles cut from stainless steel sheets. Such a cylinder with protruding brushes is installed along one side or end of an aeration basin, with the prongs or brushes sumberged about two thirds of their radius. As this cylindrical brush is turned, intense turbulence is created in the immediate vicinity of the prongs causing air entrainment and at the same time the dragging force of the brushes sets the liquid to circulating in the basin. The tank has a vertical side where the brush is mounted, a rounded bottom, and the opposite side sloped about two to one. By installing a baffle so as to direct the liquid coming to the brush from the bottom of basin, a circulatory motion can be established which emanates from the brush along the surface and then turns down at the opposite side or end of the basin and returns along the bottom and up into the rotating brush.

It is not possible at the present time to draw any reliable conclusions as to the efficiency of brush aerators with

respect to other types of surface aerators. There is considerable difference in the results presented by various authorities. It does appear, however, that brush aeration is comparable to the other types of surface aerators.

One of the most significant facts to emerge from investigations with the Kessener brush was that oxygenation values in the order of25 lbs/hr/1,000 cu. ft. which is 6 to 10 times as high as in standard aeration systems in practical use was obtained. This high value was obtained by reducing the volume of water per unit length of brush to such an extent as to be impractical for treatment plant operation. This high value is not indicative of brush aerator oxygenation capacity because this figure could probably be duplicated with other mechanical aeration systems under the same conditions. However, it does demonstrate the profound effect of turbulence and mixing on the oxygen transfer rate.

Mechanical Air Dispersers

Mechanical air dispersers are those which dissolve oxygen solely through their direct influence on sparger air. One or more turbine impellers may be used to maintain the proper distribution of power between fluid shear and pumping so as to achieve the most consistent and effective results for mixing and gas absorption in an aeration system no matter what its size or shape. Extremely high oxygen transfer rates are attainable with this type of aerator. Figures 9 & 10 show the "Vorti-Mix" aerator. Figure 11 shows the "D-O Aerator."

A sparger ring immediately below the inlet to the impeller serves the purpose of distributing the incoming air uniformly to the inner periphery of the radial blades. A powerful centrifugal pumping action is produced by the impeller or impellers and the air is entrained by the entering liquid. As it passes through the blades of the impeller and is discharged radially, the air bubbles are subjected to a powerful shearing action. The number of radial blades is never less than 12 and may be as high as 24 for large impellers. The height of the blades is normally kept between 1/3 and 1/4 of the impeller diameter. The radial length of the blades is approximately 1/6 of the impeller diameter.

Field Studies

Unusually high oxygen absorption efficiencies in the order of 38% can be obtained with mechanical air dispersers if power consumption is not an important consideration. In sewage and industrial waste treatment, operating costs cannot be ignored and thus high oxygenation rates may become uneconomical. High oxygenation rates, of course, result in reduced aerator volume, but a complete economic study would be required for a particular installation in order to determine the most suitable operating level. In general, it has been found that for sewage and industrial waste treatment, the minimum

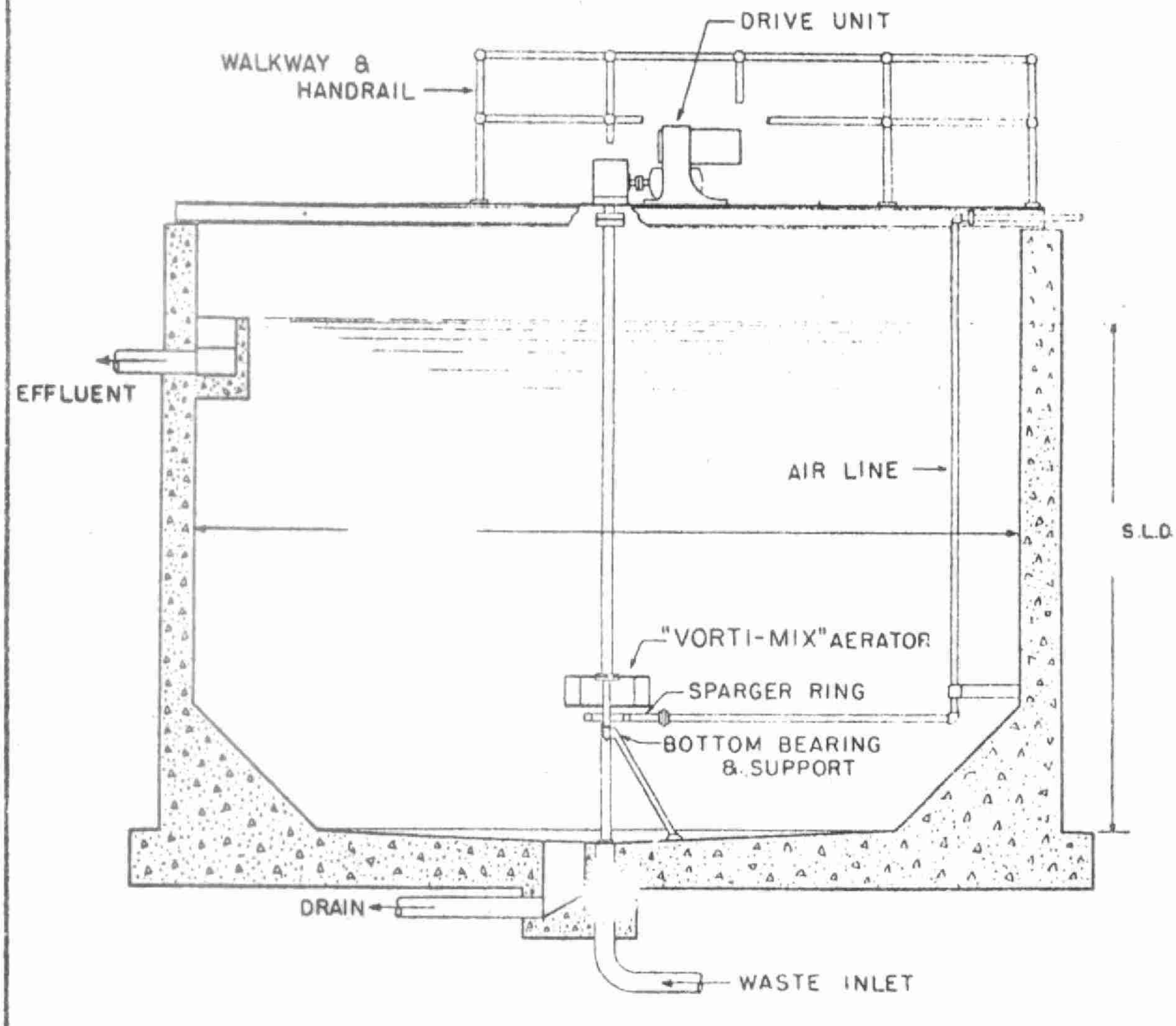

DRIVE UNIT
WALKWAY &
HANDRAIL
EFFLUENT
AIR LINE
S.L.D.
"VORTI-MIX" AERATOR
SPARGER RING
BOTTOM BEARING
& SUPPORT
DRAIN
WASTE INLET

FIGURE 9

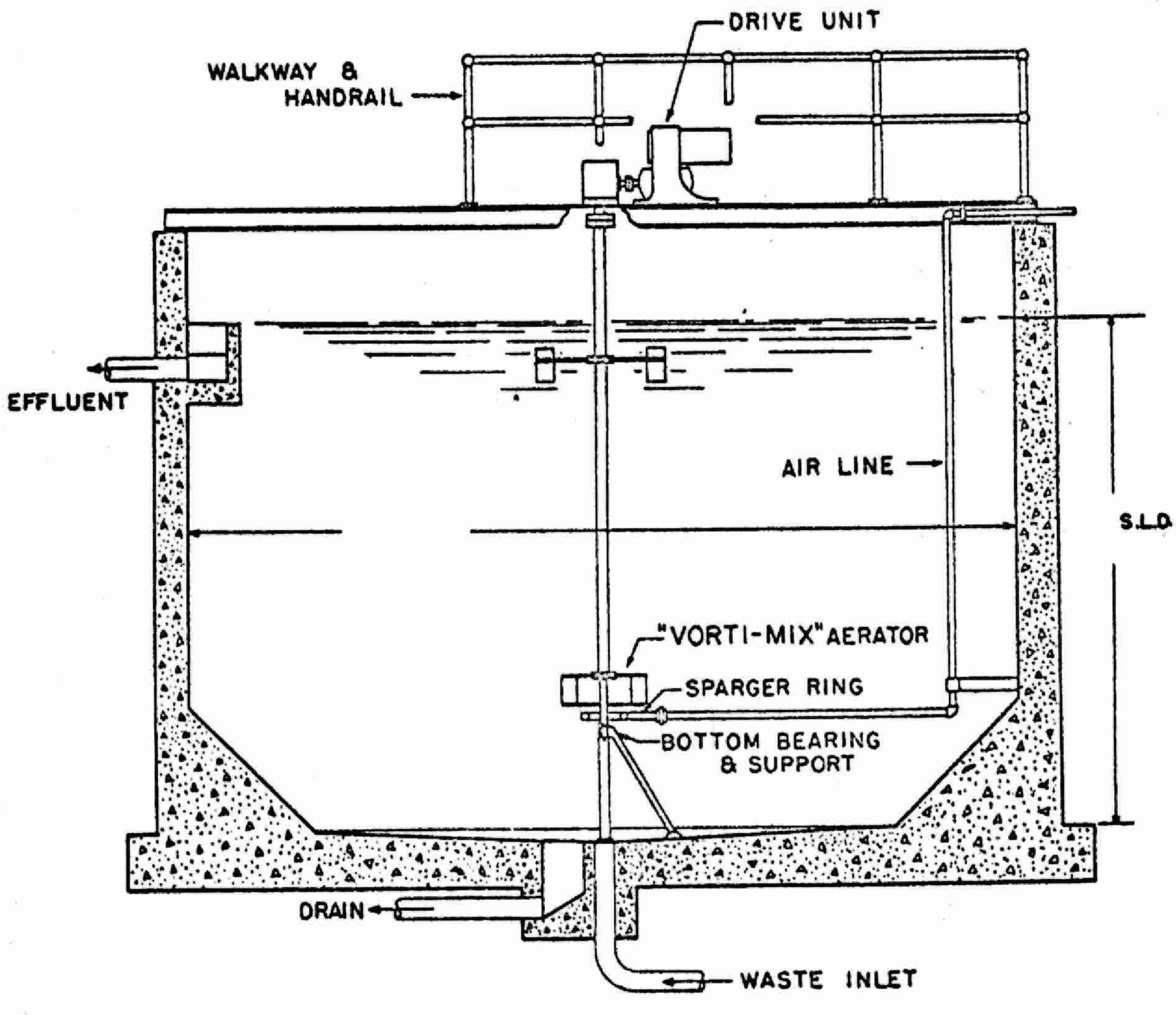
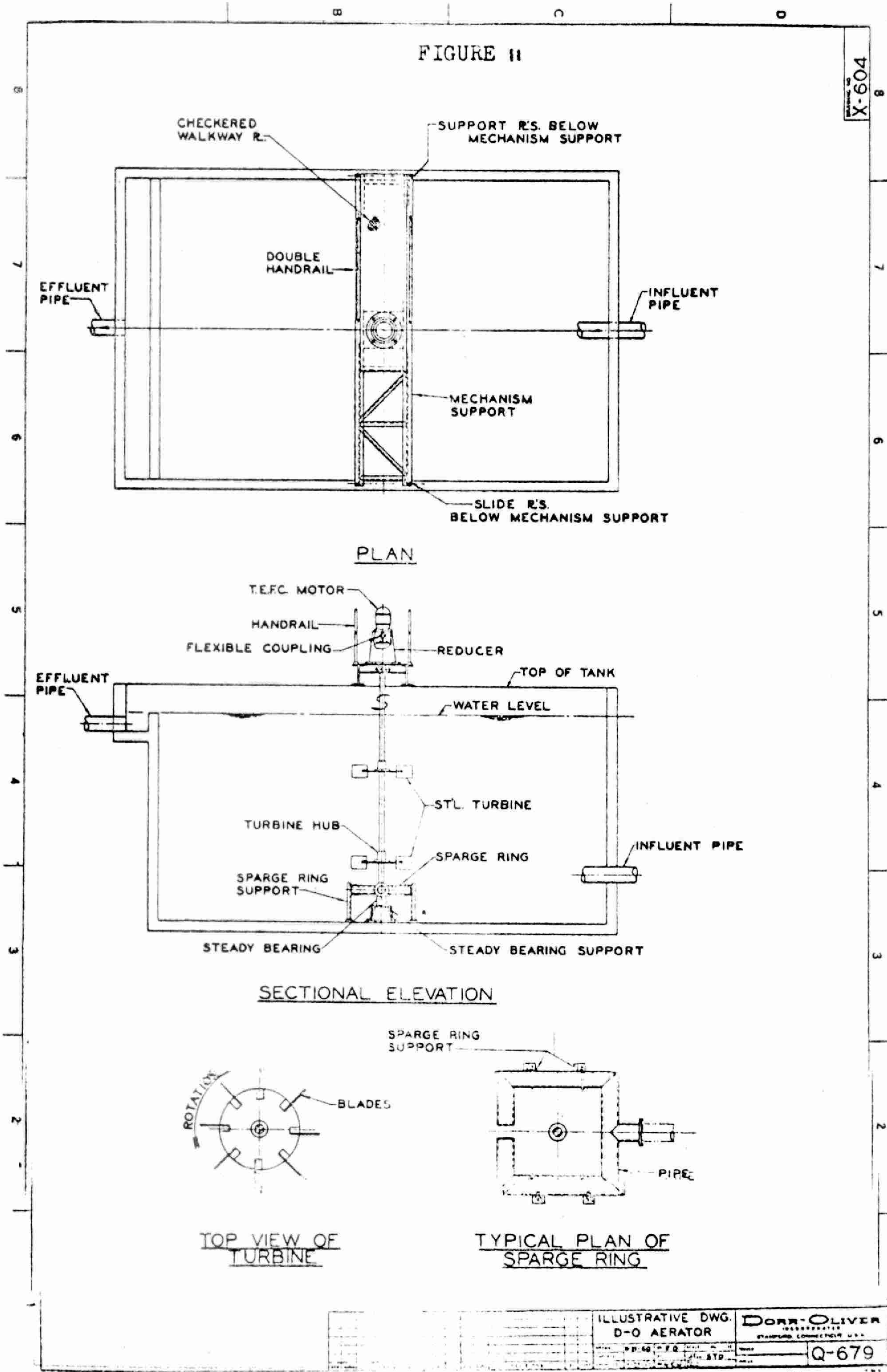

DRIVE UNIT
WALKWAY & HANDRAIL
EFFLUENT
AIR LINE
S.L.D.
"VORTI-MIX" AERATOR
SPARGER RING
BOTTOM BEARING & SUPPORT
DRAIN
WASTE INLET

FIGURE 10

FIGURE 11



X-604
CHECKERED WALKWAY PL.
SUPPORT PL.S. BELOW MECHANISM SUPPORT
DOUBLE HANDRAIL
EFFLUENT PIPE
INFLUENT PIPE
MECHANISM SUPPORT
SLIDE PL.S. BELOW MECHANISM SUPPORT
PLAN
T.E.F.C. MOTOR
HANDRAIL
FLEXIBLE COUPLING
REDUCER
TOP OF TANK
EFFLUENT PIPE
WATER LEVEL
STL. TURBINE
TURBINE HUB
SPARGE RING
INFLUENT PIPE
SPARGE RING SUPPORT
STEADY BEARING
STEADY BEARING SUPPORT
SECTIONAL ELEVATION
SPARGE RING SUPPORT
ROTATION
BLADES
PIPE
TOP VIEW OF TURBINE
TYPICAL PLAN OF SPARGE RING
ILLUSTRATIVE DWG.
D-O AERATOR
DORR-OLIVER
Q-679

power consumption will occur when the efficiency of oxygen absorption is about 10 to 15% and for efficiencies of 35% the total power consumption is about 0.24 KWH/lb (net) of oxygen absorbed.

"Cavitator" Assembly

The "Cavitator" assembly (Figures 12 and 13) consists of a vertical draft tube with a lower diffusion plate. The draft tube has openings for connection to the influent pipe and the return activated sludge line. The "Cavitator" rotor assembly is of the multiblade type, supported by an adjustable ball thrust bearing mounted at the motor level. The rotor is mounted on a stainless steel shaft and the entire unit including draft tube is supported from a structural steel bridge.

As soon as the rotor exceeds a certain critical speed, air is drawn in from the atmosphere through the vertical hollow tube to fill the areas of rarefield under pressure. The air is introduced to the cavitation zones prior to their collapse by means of nozzles at the ends of the rotor. The nozzles are claimed to be of ample size to prevent closing. The moment the air leaves the rotor it undergoes very considerable changes in pressure, the air laden voids collapse and the air contained therein is absorbed by the waste liquid. The implosion of the voids produced by the cavitation effect is such that there is considerable pressure on the material within such voids; consequently, there is a rapid absorption into the fluid waste of the oxygen from the air. A high degree of oxygen absorption is obtained because of the rapid and continuous replacement of the interfacial surfaces between the air bubbles and the body of the liquid.

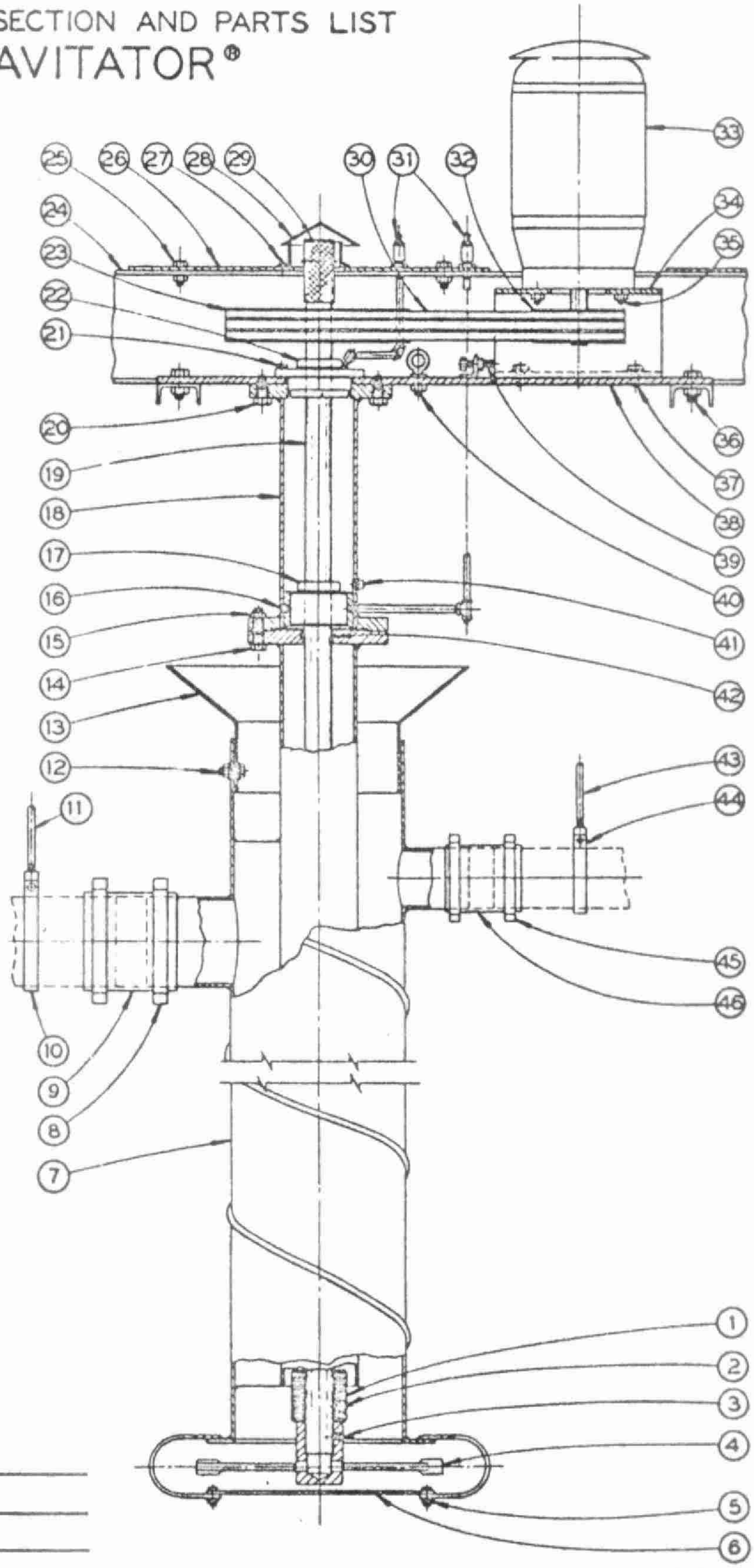
Primary effluent, mixed liquor, and return activated sludge are drawn down through the draft tube. The circular plate below the rotor directs the waste flow through the turbulent cavitation zone and thus mixes it intimately with the air. The rotor arms are designed to prevent excessive wear and to localize the regions of cavitation. The arms are streamlined to reduce skin friction to a minumum and to prevent the occurrence of cavitation in any position other than at the tip where the design is so changed to produce sufficient pressure at predetermined speeds to cause cavitation.

Since the entire unit is self contained and hangs freely from the bridge,in case of servicing, it can easily be pulled up and then lowered again. The hollow shaft is completely housed, and the housing carries water lubricated bearings. Optimum absorption rates of 129 ppm oxygen per hour per 1000 cu. ft. of air, equivalent to 4.57 lbs. per hours, were obtained with a power requirement of 0.30 KWH/lb. of oxygen absorbed.

Field tests on full scale units indicate that oxygen absorption efficiencies in the range of 45 to 50% are common.

CROSS SECTION AND PARTS LIST
CAVITATOR®

LIST OF PARTS	
NO.	DESCRIPTION
1	SLEEVE
2	SET SCREW
3	SET-SCREW
4	ROTOR-ARMS & HUB ASSEMBLY
5	BOLT, LOCK-WASHER & NUT
6	BAFFLE PLATE
7	LOWER-SUSPENSION-PIPE AND DRAFT TUBE ASSEMBLY
8	PIPE CLAMP
9	FLEXIBLE-PIPE-COUPLING
10	HANGER CLAMP
11	HANGER ROD & NUT ASSEMBLY
12	BOLT, WASHERS & NUT
13	ADJUSTABLE FLARE
14	CAP SCREW
15	STOP NUT
16	SET SCREW
17	ROLLER BEARING
18	UPPER SUSPENSION PIPE
19	HOLLOW SHAFT
20	CAP SCREW
21	CAP SCREW
22	ROLLER BEARING
23	SHEAVE
24	BRIDGE
25	BOLT & NUT
26	COVER PLATE
27	CAP SCREW
28	AIR INTAKE HOOD
29	SCREEN
30	V-BELT
31	LUBRICATION LINES-SEE DR.60500
32	SHEAVE
33	MOTOR
34	MOTOR SUPPORT
35	STUD & NUT
36	BOLT & NUT
37	CAP SCREW & WASHER
38	SUSPENSION PLATE
39	ADJUSTING BOLT
40	EYE-BOLT & NUT
41	PIPE PLUG
42	O-RING
43	HANGER ROD & NUT ASSEMBLY
44	HANGER CLAMP
45	PIPE CLAMP
46	FLEXIBLE PIPE COUPLING



S.O. NO.________ DATE:________ CERT. BY:________

NAME: ____________________________________

LOCATION:__________________________________

DRG. NO. 60315

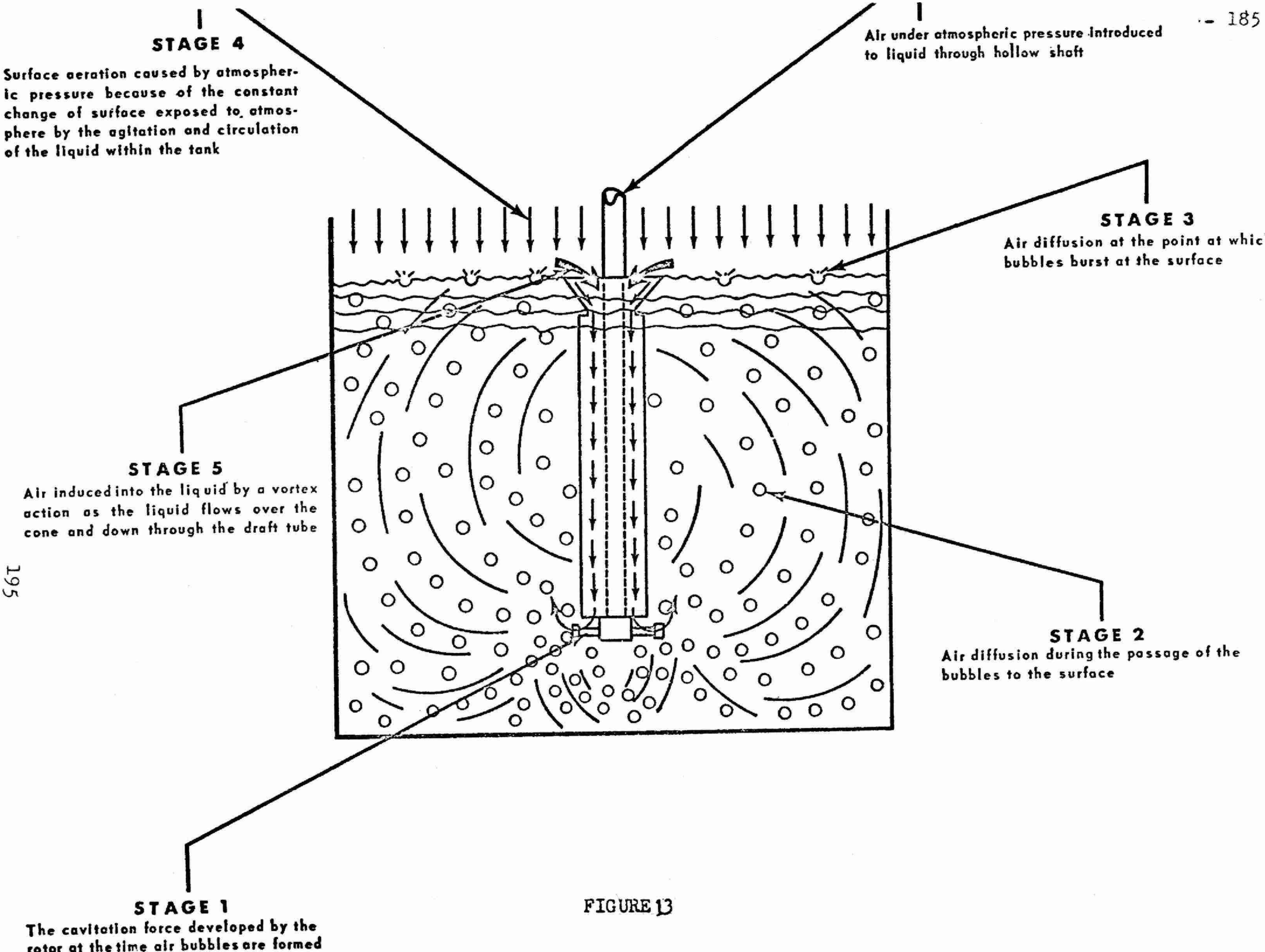

Air under atmospheric pressure introduced to liquid through hollow shaft
STAGE 4
Surface aeration caused by atmospheric pressure because of the constant change of surface exposed to atmosphere by the agitation and circulation of the liquid within the tank
STAGE 3
Air diffusion at the point at whic bubbles burst at the surface
STAGE 5
Air induced into the liquid by a vortex action as the liquid flows over the cone and down through the draft tube
STAGE 2
Air diffusion during the passage of the bubbles to the surface
STAGE 1
The cavitation force developed by the rotor at the time air bubbles are formed

FIGURE 13

MAINTENANCE AND CALIBRATION OF FLOW MEASURING AND CONTROL DEVICES

by

A. C. Beattie

Regional Assistant Supervisor
Division of Plant Operations - OWRC

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 25, 1963

MAINTENANCE & CALIBRATION OF

FLOW MEASURING & CONTROL DEVICES

by

A. C. BEATTIE

Regional Assistant Supervisor

Flow Measuring and Control Devices

An outline of the principle devices used in flow measurement and control will be given to acquaint the student of the basic theory of operation of each of the devices. Some of the basic theory was given in greater detail in the lecture "Flow Measurement and Sampling Routine" in the primary sewage works course, but will be repeated here so that a complete review can be made.

Force, length and time are quantities which can be measured very accurately. It is good technique to refer all derived measurements, such as pressure, velocity and rate of discharge, as directly as possible to the primary standards of force, length and time. For example, in order to determine the rate of discharge from a pump, good accuracy can be obtained by weighing the discharge during a measured time interval. In some cases, however, economy and convenience may dictate the use of a secondary instrument, whose accuracy depends solely on a calibration or assumed coefficients.

The formula for flow is as follows:

$Q = AV$ in which

Q = Volume, in cubic feet per unit time, T;

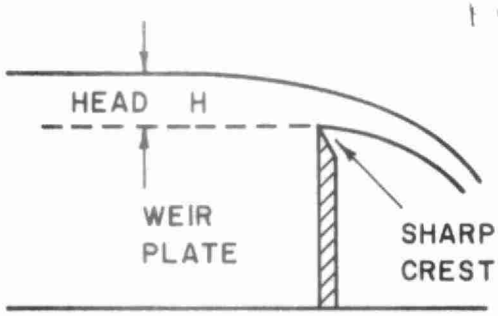
A = Area, in square feet; and

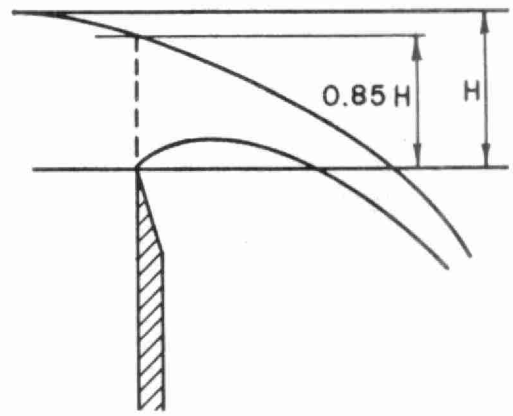
V = Velocity in feet per time unit T.

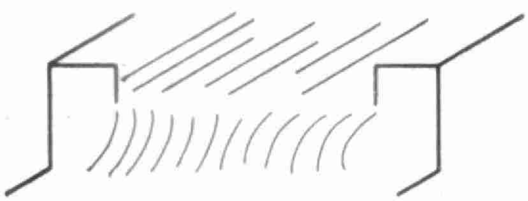
Velocity is a fundamental factor in the measurement of flow. It is this basio concept on which all flow measuring devices (except gravity scales) are based.

WEIRS

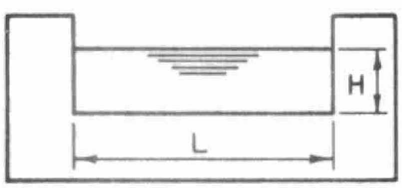
FIGURE 1



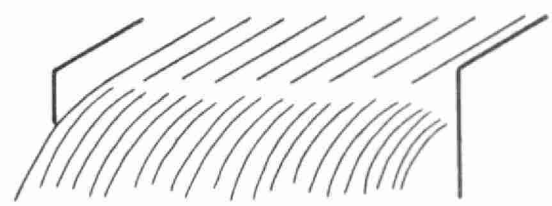




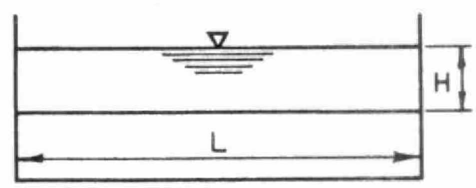
CONTRACTED WEIR



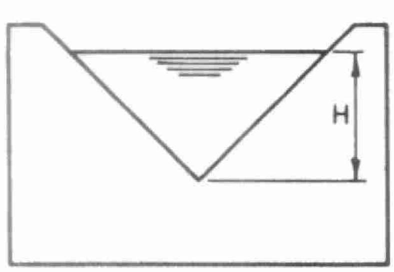
$Q = 3.33\,(L - 2H)\,H^{\frac{3}{2}}$



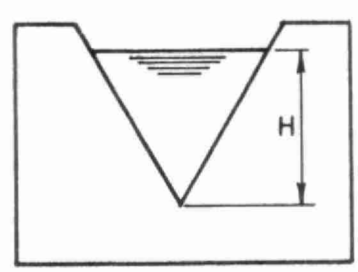
SUPPRESSED WEIR



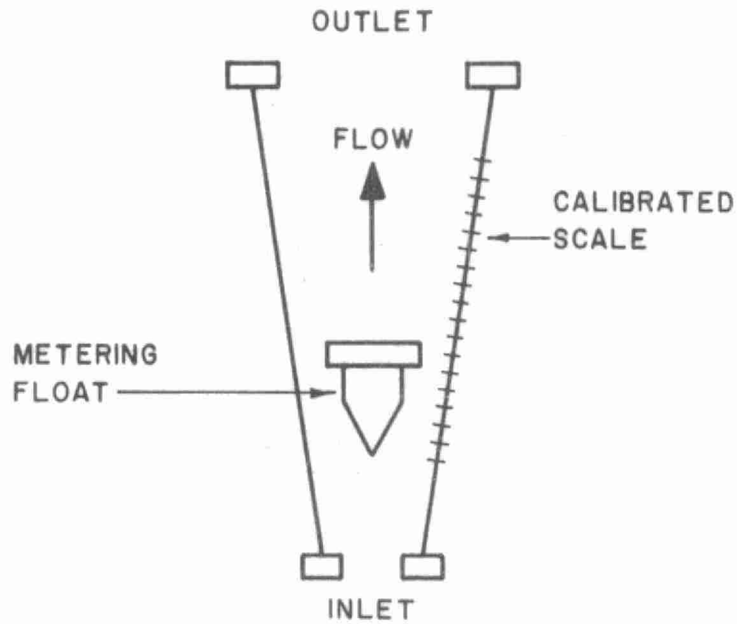
$Q = 3.33\ LH^{\frac{3}{2}}$



90° $Q = 2.52\ H^{\frac{5}{2}}$

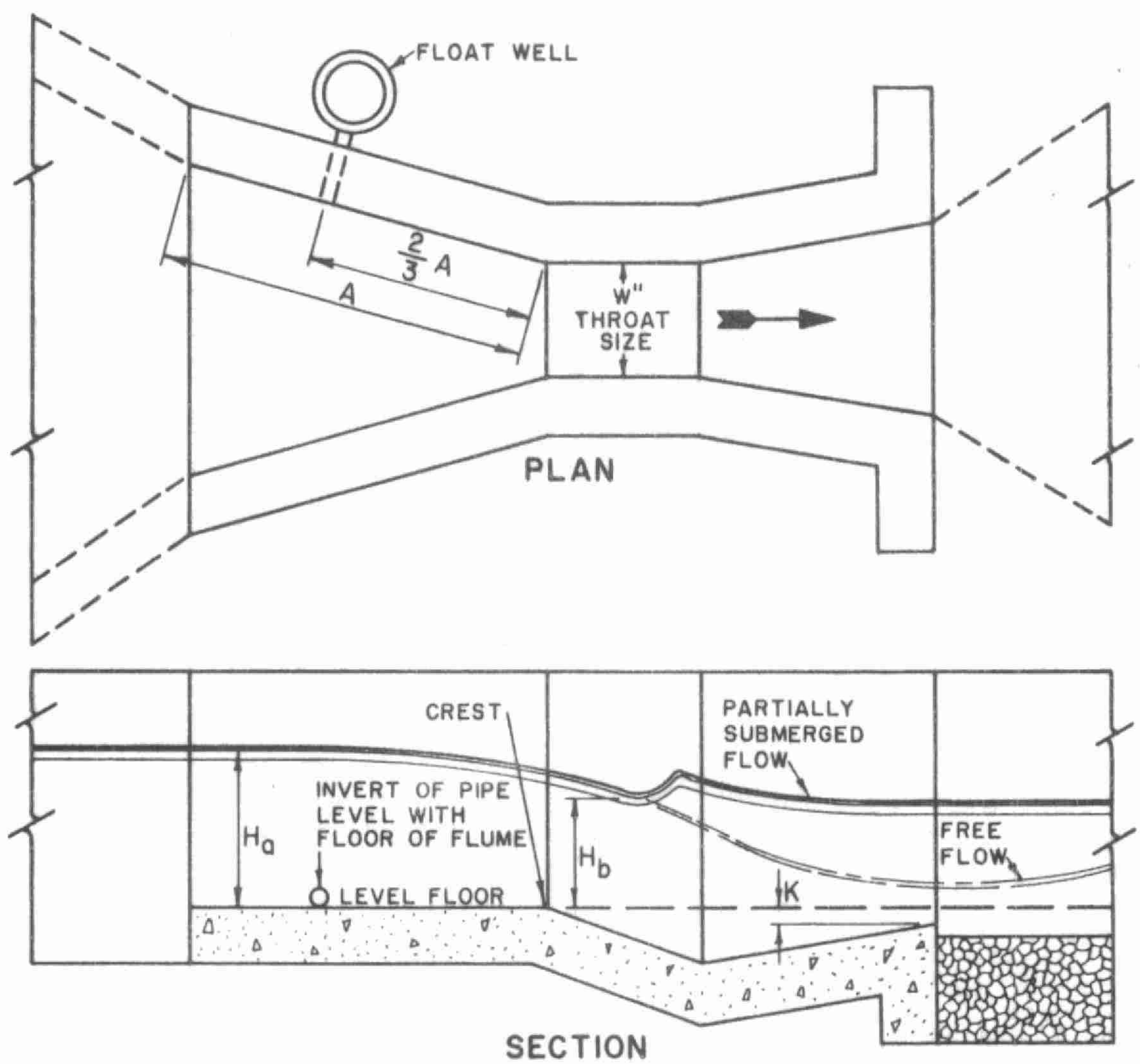


60° $Q = 1.46\ H^{\frac{5}{2}}$



ROTOMETER

FIGURE 2



PARSHALL PLUME

GRAVIMETRIC SCALES

A scale can determine the weight of fluid contained in relatively small containers, and thus give a measure of total quantity.

FILL-AND DRAW DEVICES

The total quantity of a liquid can be determined if the liquid enters and leaves a tank of known volume. The change in liquid level in the tank determines the rate of flow.

CONSTANT DIFFERENTIAL METERS

The rotometer is a rate-of-flow indicator. It consists of a tapered tube, smaller at the bottom than at the top, and a movable element, called the "float", which rises in the tube as the flow through the tube increases. The difference in pressure below and above the float remains constant and the position of the element gives a fairly accurate measure of instantaneous rate of flow.

HEAD-AREA METERS

A head-area meter is a flow measuring device used only for open channel flow. It operates on the principle that a constriction or controlled barrier in the flow channel will back up the liquid creating a higher level or head than the level below the barrier. This head or elevation of the liquid is a function of the velocity of flow and, therefore, of volume flowing through a known constriction per unit time.

Weirs

The height of the surface of the liquid above the weir crest is a function of the velocity of the liquid, and, therefore, a function of rate of flow, since the cross-section of the weir is pre-determined. The point at which this height (head, h) is measured must be upstream from the weir a distance of at least three times the maximum head to be expected.

Measuring Flumes

Measuring flumes are constricted channels of predetermined dimensions. There are two types: the Venturi flume and the Parshall flume.

Parshall Flume

In the Parshall flume, a stilling well and tap are placed in

the converging section at a point two-thirds of the length of the converging section, above the crest of the flume. The tap is at right angles to the wall and at the invert level.

The accuracy of Parshall flumes depends on the downstream elevation (head above the floor) being held to less than a definite percentage of that upstream. Under conditions of free flow, the elevation of the liquid downstream has no effect on the rate of discharge. Under conditions of submerged flow, where the downstream elevation is sufficient to cause a backing up, the rate of flow through the flume will be retarded.

The degree of sumbergence is determined by measuring two heads: Ha at the upstream tap, and Hb at a tap located 2 inches upstream and 3 inches vertically above the intersection of the plane of the throat floor and the plane of the diverging section floor.

For degrees of submergence greater than 70 percent ($Hb/_{Ha} = 0.7$) in flumes above 1 ft. throat width and 60 percent for throats less than 1 ft. wide, the observed readings will be incorrect.

FUNCTIONAL METERS

Functional meters are mechanical devices which measure some function of the volume or mass in movement, and by means of other mechanical devices convert this measurement into units of rate of flow or total quantity of flow.

DIFFERENTIAL HEAD METERS

The most popular and most accurate flow measuring devices are the differential producers. These function by means of some form of constriction in the device, which causes a change in velocity of flow accompanied by a change in pressure (head), which change bears a definiterelation to the rate of flow.

Fundamentals of Flow Relations

In order to understand the functioning of differential head meters, it is necessary to elaborate on the general concept of flow and to explain some of the fundamental relationships existing in fluid flow.

A liquid has energy (capacity to do work) because of its elevation, velocity, pressure, or any combination of these. Each of these sources of energy may be expressed in terms of equivalent head in feet (or p.s.i.) and each can be converted into the other two.

Example: 1 p.s.i. = 2.3ft.

Total Head

Total head at any point in a system is equal to the sum of the elevation head, pressure head, and velocity head. In any system a decrease in anyone of the heads causes a corresponding increase in the others so that the total head remains the same.

If a liquid flowed through a smooth pipe without friction, the sum of the three forms of energy, or total head, would remain the same at all points in the pipe, even if the diameter or elevation of the pipe changed.

The word "head is used frequently in flow measurement and the various kinds of head must be understood. Potential energy, or energy of position, of a liquid is due to its elevation above some point. The height above that point is called "elevation head".

When a liquid flows in a pipe, a part of the total head is converted into kinetic energy (energy of motion) capable of lifting the liquid through a height called the "velocity head". If there were no friction, liquid flowing through a pipe between two elevations would have the same velocity as a body falling freely between those same two elevations. The energy of velocity will lift a liquid in a Pitot tube to the height of the original elevation (less friction) or to the difference between the elevations of the liquid in Tanks 1 and 2 in Figure 4.

Pressure Head

Pressure head is equivalent to the pressure per unit of area exerted on the walls of the pipe. If is equal to the difference in elevation between the hydraulic grade line and the center line of the pipe (Figure 4).

Friction Head

Friction head is the loss of head due to friction, turbulence, pipe roughness, valves, elbows, and fittings, and friction forces when water flows from one point to another. In Figure 4, the friction head is the difference in total head at points 1 and 2. Also shown in this figure is the "hydraulic grade line", which is a line, in any selected plane, connecting all points representing pressure head in any system.

THEORY OF DIFFERENTIAL METERS

In any pipe, regardless of friction or variations in the cross-sectional area of the pipe, an equal quantity of water must pass all points during the same time interval. This is known as "continuity of quality" and is fundamental in the functioning of any differential head meter. This can be shown as $Q = A_1 V_1 = A_2 V_2$ = constant.

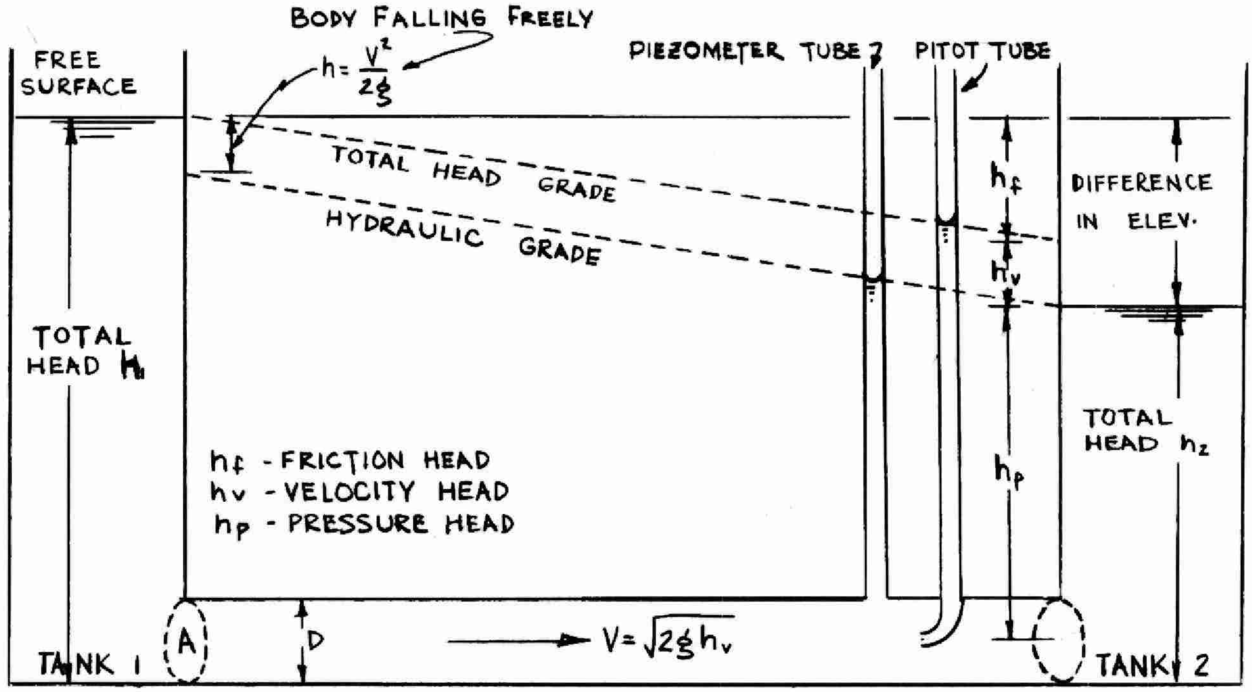
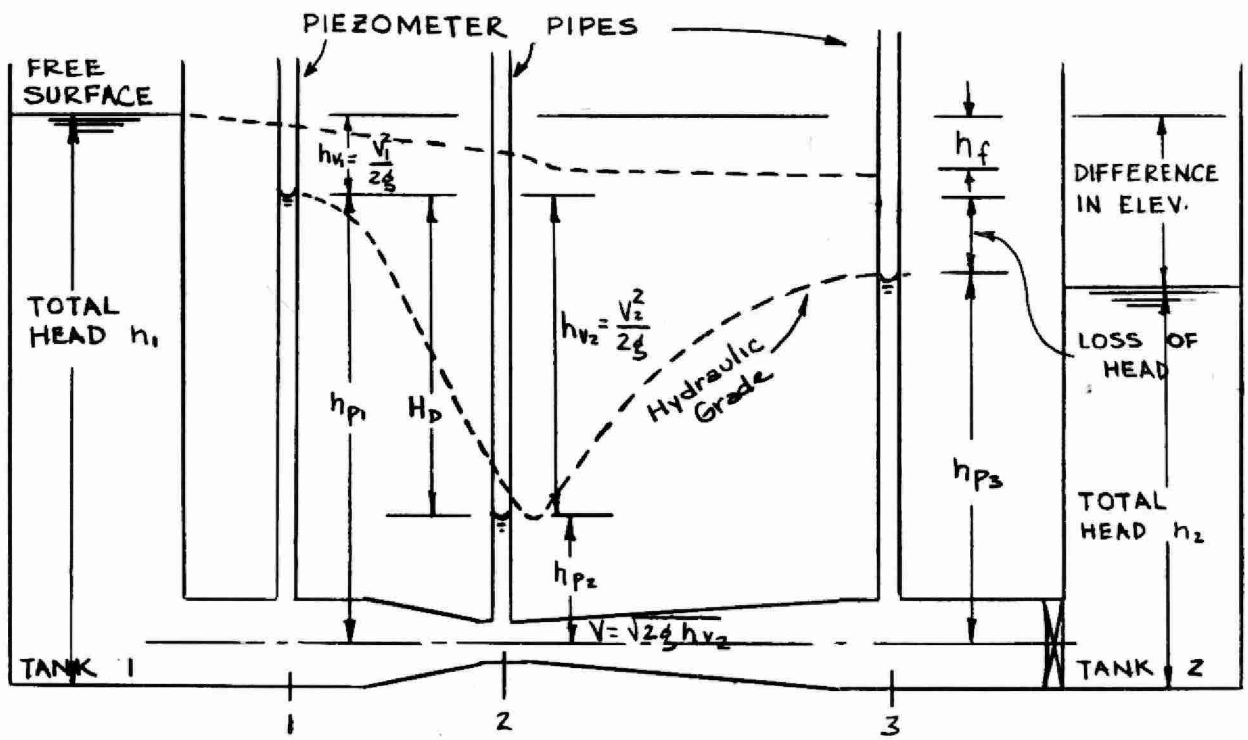


FIGURE 4
HEAD RELATIONSHIPS

THEORY OF A DIFFERENTIAL HEAD PRODUCER
FIGURE 5



If a constriction is substituted for the pipe in Figure 4, as is done in Figure 5, and piezometer tubes are installed at points 1, 2 and 3 and a valve at the outlet to Tank 2, the theory of a differential head meter may be demonstrated. With the valve closed, there is no flow and the liquid will stand in the piezometer tubes at the same elevation as the free surface in Tank 1.

When the valve is opened, the difference in elevation between the two tanks will cause flow from Tank 1 to Tank 2. Part of the total head will be converted to velocity head at Point 1. Since an equal quantity must pass both throat and inlet during the same time, the velocity at the throat will have to increase proportionately to the decrease in area. At the same time, as the velocity increases, the velocity head increases and hence pressure head decreases: the greater the velocity, the lower the pressure head at the constriction.

Therefore, the difference in pressure head or differential head (Hd in Figure 5) between the inlet and the throat is a measure of velocity, which is a measure of rate of flow, with the flow varying as the square root of the differential. The increase in velocity is equal to $\sqrt{2g\ h_v}$, where h_v equals differential head and the flow formula then becomes

$$Q = A\sqrt{2_g\ h_v}\ \ldots\ldots\ldots\ldots$$

Loss of Head

As previously pointed out, a liquid does not move from one point to another without friction; therefore part of the pressure head is lost in movement of the liquid. In any differential head producer, there will be a decrease in pressure head, but a portion of that decrease will be recovered as the flow continues on through the line. Any constriction in a line increases the friction of flow and hence increases the loss in pressure head between the inlet and outlet (Points 1 and 3, Figure 5). It is possible to construct devices which can recover a substantial portion of the head decrease at the constriction. That head which cannot be recovered is called permanent loss of head.

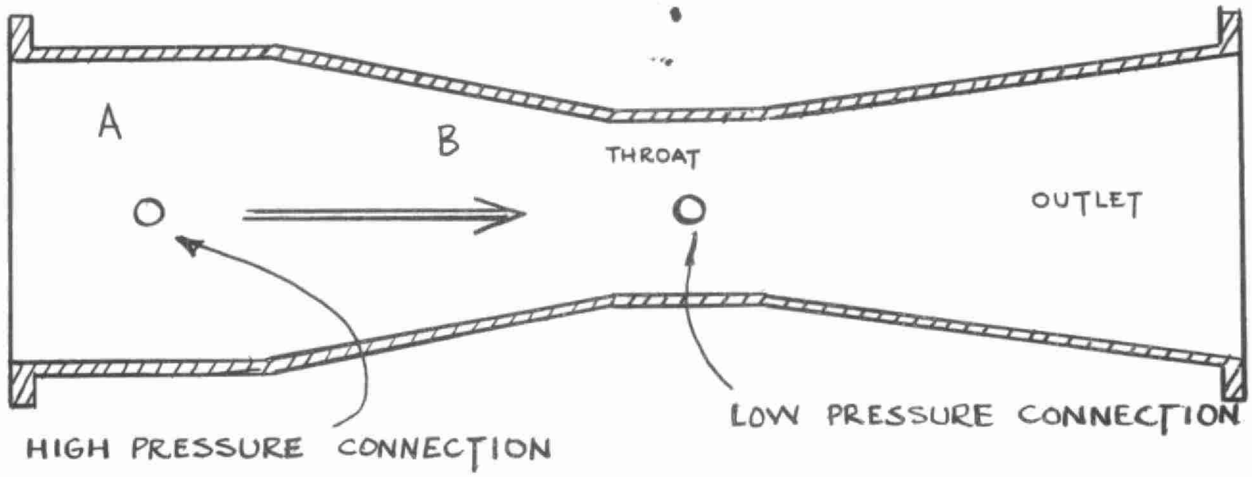
Since permanent loss of pressure head requires power to be overcome, a device which creates the lowest head loss is desirable for flow measurement. The smaller the constriction in relation to the original pipe size, the greater the head loss. Also, the smaller the constriction in relation to the pipe, the greater the differential pressure and hence the greater accuracy of measurement.

Venturi Tube as Differential Producer

The Venturi tube is essentially a section of pipeline in which it is installed to measure flow. In itself, the Venturi tube usually consists of several sections (see Fig. 6), including a conical reducing section, a cylindrical throat section, and a

VENTURI TUBE

FIGURE 6



ORIFICE PLATES

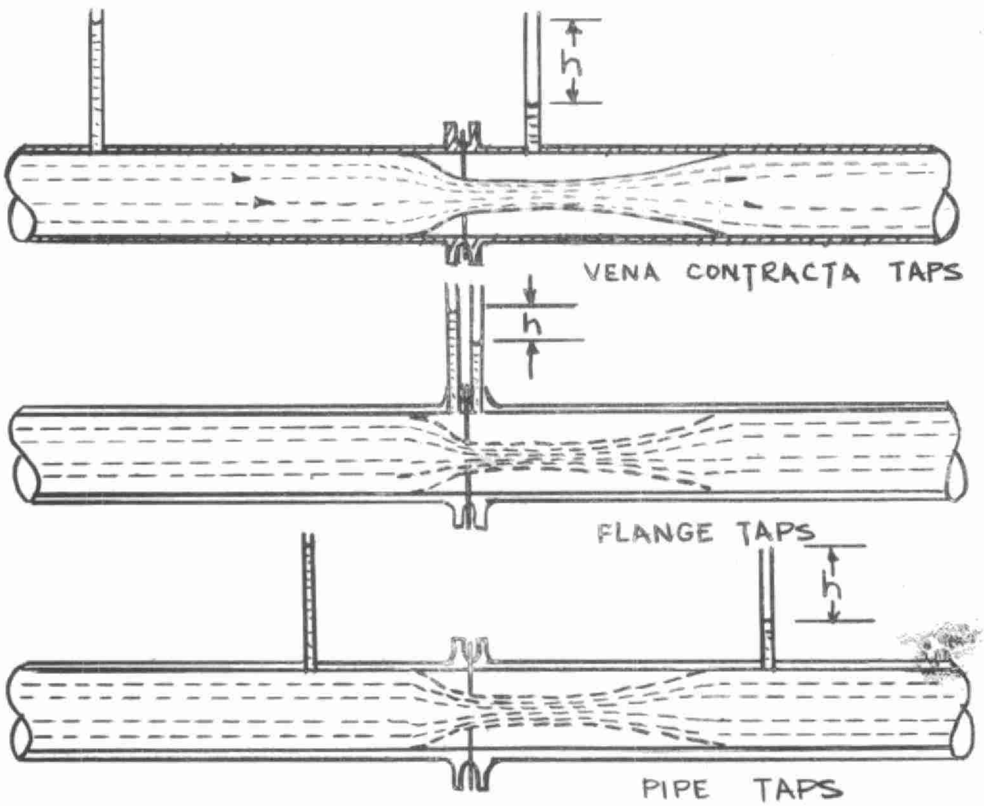
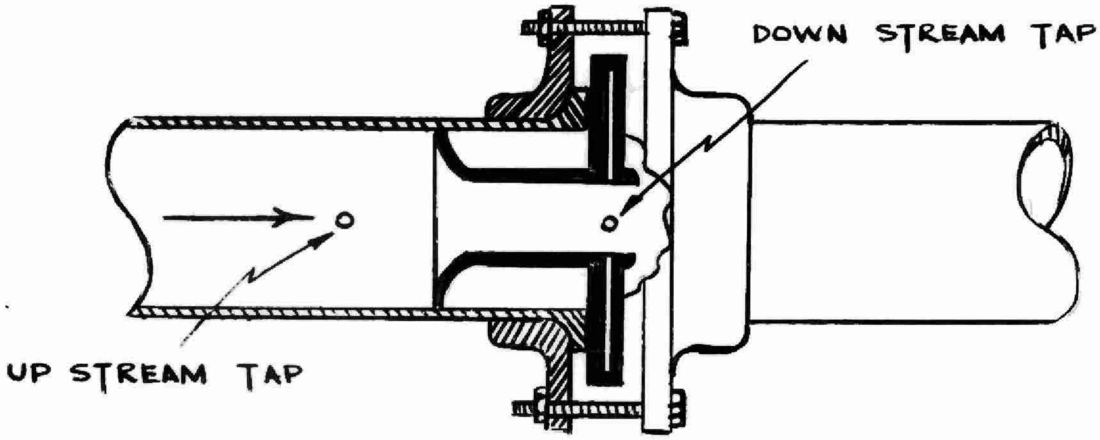


FIGURE 7

FLOW NOZZLE



FLOW TUBE

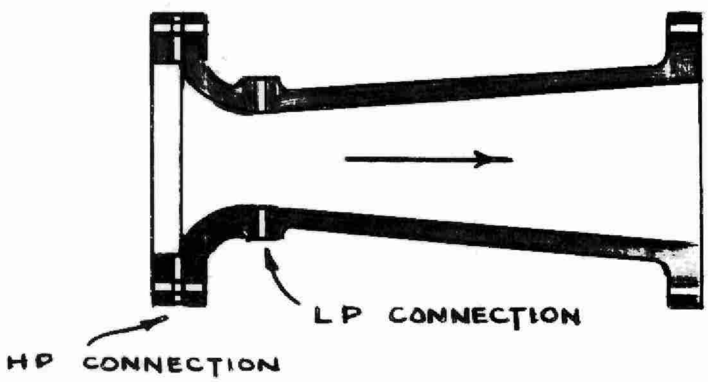


Figure 7, which also shows the pressure characteristics and the various location of the taps. There is a slight build-up of pressure just before the orifice, a sudden drop on the downstream side, and a continued drop before recovery starts. The point of minimum pressure (maximum velocity) is known as the vena contracta.

Caution is necessary in the location of orifice plates; they must be placed in a straight run of pipe with no fittings for 10 to 15 pipe diameters upstream and about 5 pipe diameters downstream. If swirls or spirals exist in the flow stream as it approaches the orifice, longer lengths of straight run pipe are required or straightening vanes must be used. (Spiral flow can be caused by elbows in different planes or by pumps and other conditions).

Orifice plates, like weirs, are in effect dams, behind which solids particles will deposit from liquids containing settleable solids.

The head loss of orifice plates ranges from approximately 40 to 90 per cent of the differential.

Insert Nozzles

Another form of constriction for producing differential head is the insert nozzle, which consists of a concentric throat section smaller than the pipe size, inserted in the pipe.

There are several variations of the insert nozzle type of differential head producer, all designed to reduce the permanent loss of head producer. Nozzles have been used for sludge measurement, but most applications of the nozzle type of differential head producer are in fields other than sewage treatment.

TYPES OF INSTRUMENTS

A short description of the advantages and disadvantages of the secondary instruments is necessary so that the inherent differences can be appreciated. Secondary instruments are those devices which convert the function of flow which is sensed by the primary device into observable information.

There are three types of information that can be obtained from secondary instruments, either separately or in any combination as follows:

1. Indication of flow rate in c.f.s., g.p.m. m.g.d., etc.

2. Running total of flow to the observed moment, in cu. ft. gal., or m.g. etc.

divergent outlet cone.

The inlet section consists of a short cylinder, A, and a truncated cone, B. The inlet section usually contains a piezometer ring or annular chamber (also called a annulus), which serves as a means of obtaining the average pressure of the fluid at the upstream point.

The throat section follows the inlet section and is the most constricted part. A piezometer ring or annular chamber also surrounds the throat section to serve as a means of obtaining the average pressure at that point.

The outlet section follows the throat section and is essentially a long truncated cone, which gradually returns to the original size of the pipe line.

The details of design and relations of one section to another are shown in Figure 6. A variation of the long tube is a short tube which has an outlet cone about one-half as long, while the length is shorter, the non-recoverable head loss is greater. It is the recovery section with its gradual return to normal pipe size which gives the high recovery of pressure head of 87 to 90 per cent found in the long form Venturi tubes.

When a venturi is to be used for metering sludge, the annular ring is eliminated and replaced by single-hole taps at the inlet and throat, and these are flushed continuously with fresh water.

Flow Tubes

A new Venturi-type differential producer is also available. Known as the Dall flow tube, it is an accurate primary device having less permanent head loss than any known type of velocity-increasing differential producer. The Dall flow tube consists of a flanged cylindrical body designed with a short straight inlet section terminating with an abrupt decrease in diameter, or "inlet shoulder," which is followed by a conical restriction and a diverging outlet separated by a narrow annular ring or throat, into which opens a single pressure-hole tap. A single-hole pressure tap also exists at the inlet. Both pressure taps are continuously flushed with clean water to prevent plugging from solids in sewage or sludge. For measuring flows of liquids containing no solids, the annular ring throat is a narrow gap, called a "slotted throat annulus."

The Orifice

The orifice is probably the oldest device for measuring or regulating fluid flow, having been used in Caesar's time. The orifice is a thin flat plate having a hole in it, inserted in the pipe.

The concentric orifice, the most common type, is shown in

3. A drawn record on a chart, showing the rate of flow as a continuous record for each instant throughout the particular (hour, day, week, etc.)

These instruments are called, respectively, indicator, totalizer, recorder. Combinations of these functions are more common than simple types of instruments

MECHANICAL

In a mechanical apparatus, the instrument is operated by cables, rods, linkages, gears, etc., which connected to pressure takeoffs on the primary device. The mechanisms are very limited in range. They depend on lengths of cables and rods which become unwieldly as distances increase. The mechanisms also become heavy and bulky with many intricate connections.

Themechanical system is not very sensitive to small changes nor does it transmit quickly. Any moving surfaces in contact can introduce friction and particular attention must be paid to bearings or pivots.

PNEUMATIC

Pneumatic control generally makes use of a varying controlled-air-pressure ranging from 3 to 15 p.s.i. Pneumatic systems have a continuous output signal (pressure) and extreme sensitivity to change in the variable being measured. Pneumatic control is simple, clean and quiet.

Pneumatic control systems have the distinct disadvantage of having a limited range of about 1500 feet. They require a clean filtered air supply, which represents a considerable capital outlay. If air is not dry, it may corrode or during the winter water in the air may freeze. Drying equipment is needed.

ELECTRICAL

Electrical control systems are capable of coping with high speed-systems. There are virtually no transmission lags to contend with. In keeping with modern trends, most of the electronic instruments and controllers have been miniaturized so that they require a minimum of space on a central control panel. Some of the controllers are of the plug-in or componentized type for ease of maintenance.

Electrical control equipment is expensive and difficult to maintain. Adequately trained personnel are simply not available in the numbers required.

OPERATING PROBLEMS

Negative Head

Negative head is a condition which may occur in a differential producer if installation is improper. It occurs when the liquid pressure inside either or both mercury wells of a differential producer instrument is less than atmospheric. When this happens, air may enter and become trapped in the pressure lines or wells, thereby causing serious errors in the readings. When the head in a pipe line is insufficient to keep a positive pressure on both wells of the instrument under all conditions, a transmitter at a lower elevation must be used, or float pipes may be used as an alternate to mercury wells, or the secondary instrument may have to be moved to a lower position.

Air in Lines

All pressure control lines should be tapped into the side walls of the primary device not into the top. Otherwise, air or sewage gas will accumulate at the high points in the line. The pressure control lines should be vented several times a year. The frequency of venting should be determined by experience in each individual case. All lines should rise to a high point which contains an air trap and a vent.

Foreign Bodies

Foreign bodies often find their way into measuring devices both during construction and during operation. Any foreign body will adversely affect operations of all measuring devices. Solids which build up behind or on the edge of weirs and orifices and cause errors in reading. The only remedy for this situation is to use care to prevent the entrance of such bodies and to remove them when they appear.

Deposits

Deposits on the surface of a primary device such as the deposition of grease of the growth of slimes on weir and orifice edges and surfaces of other differential producers will cause a change in flow characteristics of the device and readings will be incorrect. Cleaning is the remedy.

Grease also has a habit of building up in float wells or on the floats. This generally causes the float to increase in weight or stick to the sides of the float well. Sludge or sand will also plug the pipe leading to the float well or settle in the bottom of the float well and prevent the float from settling to its lowest level.

It is advisable to have purge water for all small lines in

sewage plants to prevent sewage solids from plugging the lines.

The use of purge water must be carefully controlled to prevent an artificial pressure from being created due to excessive flow of purge water through lines which are too small. Only small amounts of purge water are necessary.

For example, in one plant the purge water to the float well on a Parshall flume was increased to prevent the line from freezing during extremely cold weather. The flow was great enough to cause the level within the float well to rise above its normal level and create a higher recorded flow.

Where possible all metering points should have purge water but in remote locations where this is not possible the device must be carefully checked and cleaned at frequent intervals.

Wear & Moisture

Wear on transmitter or receiver parts, especially if the wear is on a cam, will cause the apparent reading to vary from the true reading. Regular inspections should be made to prevent dirt and corrosion from creating excessive friction in the linkage system.

The air supply for pneumatic controls must be free of oil and dirt and dry to prevent the plugging up of lines and nozzles.

MAINTENANCE RECORDS

A proper preventative maintenance schedule involves the establishment of a card or book file on each piece of equipment.

The maintenance schedule should be set up to show the operations to be performed and the time of year at which the work is to be done. The book or file should be arranged so that the card for each particular piece of equipment will come to your attention automatically on the date established for maintenance work on that equipment.

Records should contain:

a) Routine inspection reports on the condition of the equipment.

b) A record of worn parts and replacements needed.

c) Any unusual maintenance performed to remedy trouble.

d) Records for cost accounting such as man hours required, cost of parts, emergency service calls, etc.

An efficient maintenance program can minimize repair and replacement charges. Where skilled personnel are available, the meter calibration should be checked periodically. Minor repairs should be made at once, but only by a competent person. An untrained person can do more harm than good.

Repairs, adjustments and calibration should only be made by those qualified. A sound maintenance schedule will include arrangements for a regular visit of a factory trained field service man, once or twice a year.

MAINTENANCE

Meters are of very little value unless they are accurate. To have good accuracy requires a proper preventative maintenance program.

Preventive maintenance is good insurance against inconvenient and costly breakdowns. The advantages of a good preventative maintenance program are that the work can be done at a time when there is a minimum of disruption to the operation of the plant and can usually be done under more favourable working conditions with the proper tools and parts necessary to do a proper job.

Remember, all equipment needs maintenance and good maintenance involves proper knowledge, tools, and parts for its proper performance. Preventative maintenance means that equipment must have reasonable attention at regular intervals.

SAFETY

An electrical current of 50 milli ampes through the heart or lungs can cause death. Always remove the power from the equipment before working.

INSTRUCTION MANUALS

Any and all instruction manuals and manufacturers literature must be studied as a first step toward an understanding of the construction, operation and adjustment of equipment

SPARE PARTS

Spare parts recommended by the manufacturer should be on hand to insure prompt repairs and continuous operation of the equipment.

Spare parts and lubricants should be carefully labelled to insure that they are properly used on the right equipment.

INSTRUMENT CALIBRATION

There are two basic steps in the calibration of transmitters and receivers that will be described. The first step is to compare the indicator on the transmitter to definitely known flows through the primary measuring device. Usually the most convenient points to compare are the zero point (no flow), some point midway in the meter range and the full flow point,

In figure eight a typical set-up for checking the calibration of a parshall flume is shown.

CHECKING THE INDICATOR

1. Shutoff valve (4)

2. Open drain valve (1)

3. Close drain valve (1)

4. Open valve (2) and allow purge water to overflow at B which is at the same elevation as A, the floor of the flume. The float should then rest on its saddle and the flow meter should indicate zero.

5. Valve (2) should be closed and valve (3) should be opened and the purge water should raise the level to some value mid-way in the flow rang ie: 15 inches on the clear plaster or glass tubing. The flow meter should then indicate 2.3 MGD

6. Another check should also be made at the top of the flow range of the meter.

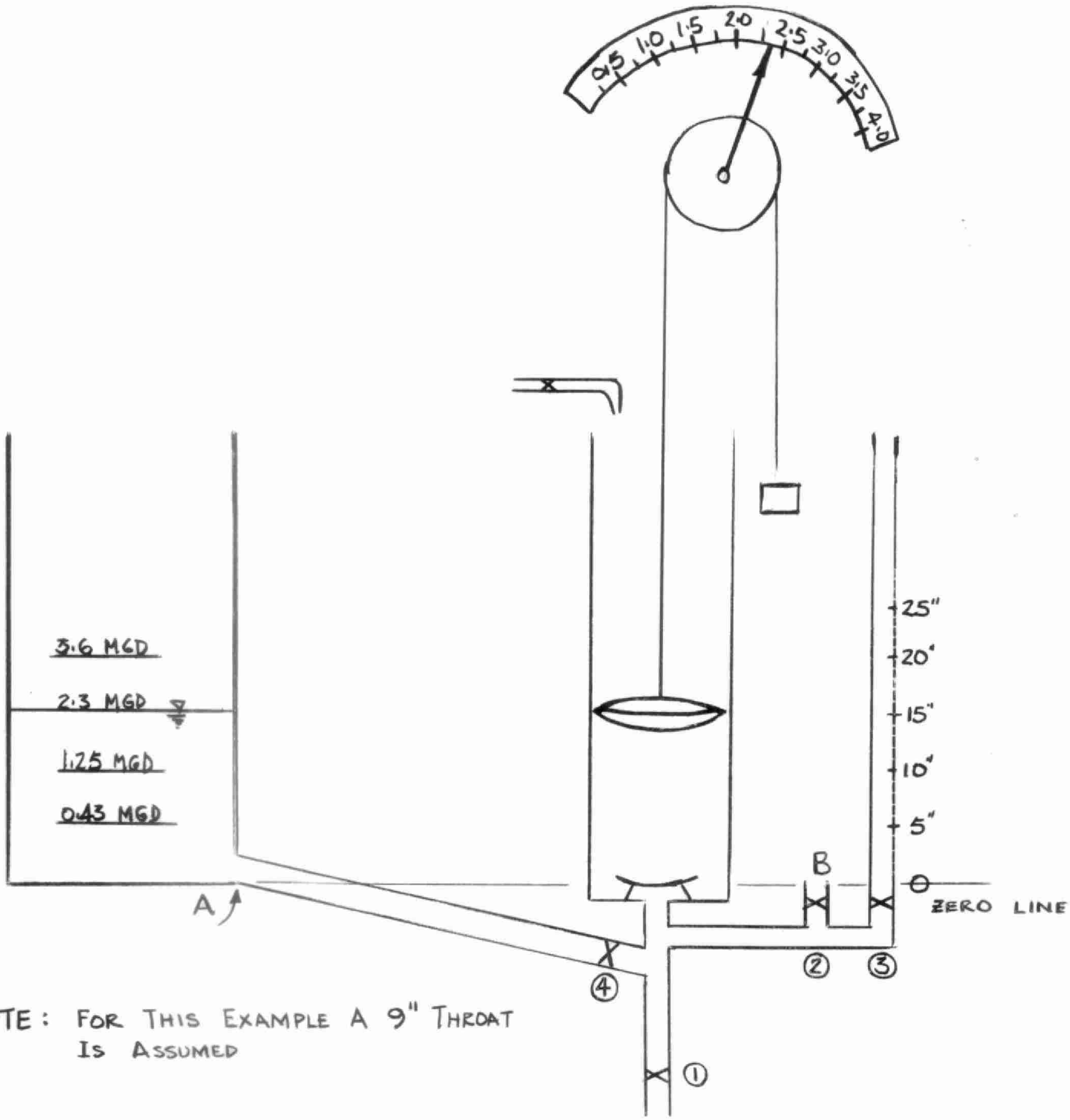
The next step is to compare the indicator with the totalizer. One simple method of comparison is to check the two functions at $0 - \frac{1}{4} - \frac{1}{2} - \frac{3}{4}$ - full scale.

For example, if we had an indicator that went from 0 to 10 MGD we would set the indicator to the zero point (as described previously) and watch the totalizer. The totalizer should not be running. We can now check the $\frac{1}{4}$ point by setting the indicator at 2.5 MGD and counting the gallons accumulated on the totalizer for 10 minutes.

$$2.5 \text{ MGD} = \frac{2,500,000}{24 \times 60} = 1740 \text{ gallons per minute}$$

If the instrument numbers record in 100's of gallons the totalizer dial should turn over 17.4 numbers during the 10 minutes of observation.

The indicator can now be set for the $\frac{1}{2}$ point of 5.0 MGD



NOTE: FOR THIS EXAMPLE A 9" THROAT IS ASSUMED

FIGURE 8

CALIBRATION OF A PARSHALL FLUME

per day and in the same 10 minutes we should observe twice as many revolutions of the totalizer dial - (ie: 2 x 17.4 = 34.8) indicating 34800 gallons.

The same procedure can be followed for the $\frac{3}{4}$ point where we would get 3 x 17.4 = 52.2 numbers or 52,200 gallons in 10 minutes which is equal to 7.5 MGD. At the top of the flow range the totalizer dial would turn 4 times as fast and give 4 x 17.4 = 69.6 number charges or 69,000 gallons during the 10 minute or 10 MGD.

The above example explains the basic theory of checking the totalizer against the indicator. In actual practice the equipment manufacturer in their maintenance instructions, give time intervals which give simple number to deal with and enable you to check the calibration without any elaborate calculations.

Before the calibration of a remote receiver with a transmitter at the primary device is described, it might be well to repeat the sequence of checking the instruments.

1. The accuracy of the primary device is checked.

2. The indicator on the transmitter is checked against specific and accurate flows.

3. The totalizer is checked against the indicator.

The accuracy of any recording chart can be checked merely by ensuring that the recording pen is indicating the same flow rate at any time as the dial indicator.

The calibration of a remote receiver with a transmitter is a difficult but not impossible task. Without elaborate equipment such as the Metameter, which will be described later, the procedure is as follows:

Two persons must have a planned procedure to follow. Having checked the transmitter as described previously, the man at the transmitter would then repeat the entire calibration according to a time schedule. If both the man at the transmitter and the one at the receiver have synchronized their watches the remote receiver should follow the same planned changes as the transmitter.

If for example is was decided to repeat the procedure described previously at 10 AM one morning, the man at the receiver should observe that exactly at 10 AM the indicator should drop to zero and remain there for 10 minutes and during that time the totalizer should not turn. After a planned time necessary to raise the water level to $\frac{1}{4}$ scale the remote totalizer should turn over the same 17.4 numbers as found before. Similar checks can be made

at the $\frac{1}{2}$, $\frac{3}{4}$ and full scale points.

CALIBRATION USING A METAMETER

A metameter is merely a portable combination transmitter, receiver and precise timer that is used in checking time pulse signals between transmitters and receivers without using the two persons with synchronized watches as described previously.

At the transmitter the metameter will check the signals received from the transmitter. The transmitter will have to be calibrated with the primary device as described previously. With the precise timing device in the metameter the exact length of the time signal can be checked.

The metameter can also be used to transmit precisely timed signals to the receiver in order to check the calibration of the receiver. Metameter signals are adjusted as a per cent of time so that it can be adjusted to various time impulse intervals.

TRENDS IN SEWAGE TREATMENT PROCESSES

by

J. R. Barr

Assistant Director & Supervisor

An Address To
The Ontario Water Resources Commission
Senior Sewage Works Operators' Course
Toronto, Ontario
April 26, 1963

TRENDS IN SEWAGE TREATMENT PROCESSES

by

J. R. BARR

Assistant Director & Supervisor

The purpose of this paper is to review some of the current trends which are occurring in the design of modern sewage treatment plants as well as some of the newer developments in sewage treatment processes, which may presently be in the experimental stage, but appear to offer definite advantages in future sewage treatment works.

In Ontario, most sewage works systems now being constructed provide secondary treatment. Due to stream requirements and the relatively low stream flows, some plants have found it necessary to provide higher degrees of treatment known as tertiary treatment to avoid injury to the receiving stream during critical flow periods. The activated sludge process is used almost predominantly in this province whereas in the smaller communities, waste stabilization ponds are being favoured. In Ontario, there are a total of 156 municipal sewage treatment plants which may be divided into the following types of treatment: activated sludge, 79; primary treatment, 32; modified activated sludge, 11; trickling filters, 16; and waste stabilization ponds, 18.

SCREENING

The development of mascerating devices known as comminutors, barminutors as well as mechanically cleaned screens has produced a marked reduction in the use of manually-cleaned bar screens in many sewage treatment plants. The simple bar screen, which is hand-cleaned with a rake, is still found in some of the small plants but even in these installations, the comminutor is more common since the removal and disposal of screenings is somewhat offensive and is often neglected. However, bar screens or trash racks are still used in even some of the larger plants on by-pass channels, overflows, etc.

The use of grinders or pulverizers, which return the materials to the sewage flow, is more common now to eliminate the need for the burial of screenings. These devices are found most often in the larger plants.

Time and level switches, which operate mechanically-cleaned bar screens or comminutors and barminutors intermittently, have been developed to reduce the maintenance and repair of these devices. This is accomplished by reducing the running time of the unit, thus reducing the wear on the cutting parts.

Recent developments include a straight-rack type of mechanism which is equipped with a travelling comminuting device. A machine of this type combines the advantages of the conventional comminutor with the simplicity of a bar screen.

GRIT REMOVAL

The provision of grit removal facilities is considered necessary in most plants, but particularly those installations which serve combined sewer systems. Even in new sewer systems, the amount of grit during initial operation of a plant may be considerable. The removal of this grit will reduce wear on sewage and sludge pumps and will prevent its deposition in sedimentation tanks, sludge lines and digesters.

There has been considerable discussion in regard to the location of grit removal facilities with respect to screening devices. The most common position is downstream of the comminutor although in this location, there is the problem of increased wear on the comminutor teeth. In some of the larger plants, it is common practice to locate the grit removal facilities upstream of the plant raw sewage pumping station.

Conventional grit channels with proportional weirs are still found in the small plants. A newer development is the aerated grit removal chamber. Air is applied in such a manner to give a velocity of roll that will not pick up grit particles which have settled to the bottom of the chamber, but will maintain the organic material in suspension. Another advantage of this type of device is the provision of some pre-aeration to the sewage flow. Grit may be removed from these chambers by a clam shell bucket, air lift or jet pumps.

The detritor is still probably the most commonly used device for grit removal in medium and large sized plants. The detritor is basically a short period retention tank with a rise rate sufficiently low enough to permit grit to settle, but high enough to keep the organic material in suspension. The grit which is deposited on the bottom of the tank is discharged to a hopper by mechanical scrapers and removed by a reciprocating rake. The grit

is then discharged to a washing or classifying device which returns the organic material to the influent end of the detritor.

SEDIMENTATION

Modern sedimentation or settling tanks are designed to operate on a continuous flow basis. Sedimentation chambers may be rectangular, circular or square in shape. The selection of the shape of tanks is generally the preference of the consulting engineer but in some cases this is governed by the topography of the plant site.

The design is based on several factors including detention, surface settling rate and weir loading rate.

The rectangular sedimentation tank, which is commonly found in the larger plants, is equipped with a straight-line sludge collector mechanism. One of the difficulties with this type of mechanism is the maintenance of underwater chains. This problem is avoided in some units by the use of a supporting bridge spanning the tank width and running on rails with rakes hung from the bridge to move the sludge from the tank bottom to the sludge hopper. However, these mechanisms are more costly than the conventional types and in cold climates, there is a tendency for ice to form on the rails and cause the mechanism to lose traction.

Circular settling tanks have the advantage of arranging multiple units in groups with a centrally located feed and effluent chamber. The most widely used sludge collector mechanism in circular tanks is the revolving device with radial arms having ploughs or blades set at an angle whereby the sludge is moved towards the centre hopper as the mechanism revolves. In such a tank, the inflow is taken to a stilling well in the centre of the tank and the effluent is discharged over a peripheral weir. A recent innovation in circular tanks is the discharge of the inflow in a spiral motion to the periphery of the tank from which it is conducted to the bottom by a baffle with the effluent weir being located towards the centre of the tank. This is known as the spiraflow clarifier and a typical installation exists at the Orangeville plant.

In recent years, the suction type collector for sludge removal in secondary sedimentation tanks has been favoured. In this collector, wide nozzles connected to horizontal pipe headers progressively cover the entire tank floor in a single revolution of the mechanism and sludge is removed from the floor of the tank by the difference in head between the surface and sludge outlet. This permits a more rapid removal of sludge but is not used in primary settling tanks. A mechanism unit, which combines both the removal of sludge by suction and its removal by rotating ploughs, has also been developed. In such a tank, the sludge

returned to the aeration tank is taken from the suction pipe header whereas the sludge that is wasted is taken from the hopper associated with the ploughs.

The principle of the Imhoff tank, which combines settling and sludge digestion, is used in modern units known as the "clarigester" and the "spiragester". These consist of a circular clarifier with a separate sludge digester compartment. Such units are of value in compact sewage treatment plants serving small installations.

Another type of combination unit includes grit removal and sedimentation in a single tank. This contains a supplementary trough in which the velocity is reduced to such an extent that grit settles before entering the stilling basin and the settling compartment. Such a unit is of value in the smaller plant in which the grit is relatively low.

TRICKLING FILTERS

In Ontario, the trickling filter method of sewage treatment has almost been abandoned with only a few plants remaining, the largest installation existing at Oshawa. Generally, trickling filters are classed as low rate or high rate filters which refers to the rate of application of the sewage flow. The high rate filters are dosed continuously at rates approximately six times those of standard filters and usually employ some form of recirculation so that the sewage passes through the filter more than once. Trickling filters may be operated in a variety of ways and numbers innovations have been developed by the various manufacturers by varying the type of recirculation employed. Many combinations of filter units and clarifiers are possible in multi-stage installations.

An outstanding advantage of a trickling filter is its ability to withstand shock loadings of industrial wastes and wide variations in sewage quality and quantity. However, several operating difficulties have been encountered which has possibly attribute to the lack of widespread use in this province.

ACTIVATED SLUDGE

The use of the activated sludge process for secondary treatment is the most predominant method of treatment in small, medium and large size installations in Ontario. The design method is based upon the relationship of the total activated sludge solids is within the range of the conventional activated sludge process whereas B.O.D. loadings above 100 lbs. per day per 100 lbs. of sludge solids indicate the high rate or modified process.

There are a number of modifications to the activated sludge process which are in common use in modern plants today.

Tapered Aeration

Aeration devices are sometines located in varied concentrations along the aeration tank length to supply oxygen in proportion to the oxygen demand. Therefore, the maximum concentration of aeration devices is found at the inlet end and then tapered towards the outlet end.

Step Aeration

In the step aeration process, the activated sludge is returned to the aeration tanks and reaerated and the sewage is then applied to the sludge in steps instead of in a single feed. In this system, the effective aeration time is greatly increased and the loading can also be much greater. The advantage of this process is its effectiveness in the handling of shock loadings because of the large amount of activated sludge in active condition for mixing with the incoming sewage flow.

Total Oxidation

The total oxidation process can either be designed as a continuous system in which the aeration detention period is usually 24 hours or as a batch fill-and-draw system. The batch system is most adaptable to industrial wastes in which the flow is received over an 8-12 hour period. The use of the total oxidation process has been employed in the initial start-up of a new plant in which the sewage flow is quite low. This may be accomplished by by-passing the primary units directly into the aeration section. The advantages of this system are high B.O.D. removals comparable to the conventional activated sludge process, and minimal sludge handling facilities.

Sludge Reaeration

In this process, sewage is aerated with stabilized sludge for a short contact period, usually 30 to 60 minutes. The mixed liquor is then separated by sedimentation and the settled sludge is transferred from the clarifier to a sludge stabilizer where aeration is continued to complete the oxidation and to prepare the sludge for B.O.D. removal from fresh incoming waste. Sludge reaeration, bio-sorption or contact stabilization processes have been employed at a number of plants. High B.O.D. reductions have been reported and it is common practice now to make provision in the design of all modern plants for the use of this process.

The oxygen required for treatment is supplied by either diffusing air through the sewage or by mechanical means and surface aeration. In diffused air plants, the spiral-flow type of tank is used almost exclusively in Ontario. Various aeration devices appear

to be effective although, generally, coarse bubble aeration does tend to be less efficient than fine bubble aeration.

In recent years, there has been a trend to mechanical aeration in the activated sludge process. The advantages of this equipment is its absence of blowers, high efficiency, lower initial and operating cost, etc. The most common mechanical aerator is the impeller type with the draft tube although recently turbine type aerators have been developed. Comparative costs with diffused air plants are not readily available as yet but these results should be interesting.

The extensive use of detergents has resulted in the need for foam control devices at sewage treatment plants, the most common of which is the spray system utilizing plant effluent.

One recent innovation in aeration equipment is the Inka aeration system, which comprises a stainless steel aeration grid about 3 feet below the surface and a fiberglass baffle wall extending along the length of the tank to produce maximum circulation of the tank contents. The air is supplied by centrifugal fans which have the advantabe of variable output. A typical installation and the first in Ontario is at Simcoe, although the results of this are not known as yet.

CHLORINATION

The most widespread use of chlorination at modern sewage treatment plants is for effluent disinfection. The trend in the design of plants today is to provide flexibility in chlorination control by installing several chlorine application points. These may include plant influent, return activated sludge discharge, supernatant return, by-pass flows, etc. The addition of chlorine is useful in odour control at various points at the plant, facilitates primary settling, controls sludge bulking and corrects filter ponding.

The use of 150 lb. chlorine cylinders is the most common at small and medium size plants with ton cylinders and even tank car containers employed at the larger installations.

Manual control of chlorinators is generally found, although fully automatic machines are available in the larger plants. The use of plant effluent as the source of water supply for chlorinators is now common practice.

SLUDGE DIGESTION

A general rule for plants designed to handle less than 1.0 MGD is to provide one primary digester and for plants greater than 1.0 MGD, at least 2 tanks are supplied with the second digester used

for concentration of the digested sludge and the formation of supernatant.

Basically, two types of roofs are used on digestion tanks, namely, fixed and floating. These roofs are normally of steel or reinforced concrete with preference for steel roofs in the modern plants. Floating covers are usually of steel and are equipped with a gas holder providing for gas storage beneath the floating cover. All tank covers are also provided with pressure and vacuum relief valves and flame traps.

The common trend is towards heated digesters with the common source of heat being fuel oil, natural gas and manufactured gas. The method of applying heat to digesters is normally by an external heat exchanger.

Accumulations of scum at the surface of sludge digesters is reduced by the provision of mixing devices of the mechanical type or gas mixing. Various mechanical devices are available as well as numerous types of gas mixing systems.

Because of the high cost and low efficiencies associated with the digestion process, there have been investigations of the use of higher loadings and shorter retention times. This has resulted in the development of the high rate digestion process which has indicated that a 15-day retention period under high rate conditions is approximately equal to that obtained in 35 days under conventional conditions. One of the fundamental requirements of high rate digestion is the need for thorough and continuous mixing of the tank contents.

SLUDGE DRYING, DEWATERING, DISPOSAL

There has been a general reluctance to provide sludge drying beds for the medium and large sized plants except possibly for emergency purposes. The general trend is toward liquid hauling of digested sludge although the cost of trucking is quite variable and may not be the most economical method of disposal at all locations.

Sludge lagooning has been used in some plants with good success but it is essential that adequate isolation from housing is provided. One of the largest installations utilizing sludge lagoons is at Ottawa and it will be interesting to observe the success at this plant.

The development of the coil spring filter has made possible the successful filtration of raw mixed sludges. However, although vacuum filtration of raw sludges is being carried out in a number of installations, there is insufficient operating data available to determine whether the disposal of raw sludge cake is an entirely

successful operation. It has also been found necessary to provide sludge storage facilities at plants where raw sludge filtering is practised to compensate for mechanical maintenance, equipment failure and to eliminate weekend operation.

There have been several recent developments in sludge handling including sludge pressing and centrifuging, AST process and the Zimmerman process. These processes have been mostly experimental to date although it would appear that the two latter processes seem to have some merit but some practical operation is needed to indicate the overall feasibility of these methods. It seems in any case that these processes are not challenging to the present sludge handling methods.

Drying and incineration of sludge has been confined to the very large plants with only two such installations in Ontario at Metropolitan Toronto and London. This may receive further consideration by the larger centres due to the difficulty of disposing of sludge on the land.

WASTE STABILIZATION PONDS, AERATED LAGOONS, OXIDATION DITCHES

The use of waste stabilization ponds for small and medium sized communities has been favoured in Ontatio in recent years. It has been found that this type of treatment is comparable to that provided by conventional mechanical type plants and generally, capital and operating costs have been substantially lower. In Ontario, lagoons are designed on the basis of one acre per 100 persons.

An aerated lagoon may be defined as a lagoon in which aerobic conditions are maintained by mechanical or diffused aeration. While the activated sludge process employs high biological solids in short aeration periods, the aerated lagoon on the other hand requires long detention periods and employs very low solids concentrations. Retention periods are normally in the range of 4 to 7 days and B.O.D. removals of 75 to 85 percent in summer and 50 to 60 percent in winter have been reported. Various aeration devices are available including turbine type aerators and conventional blowers supplying diffused air through plastic lines. The aerated lagoon has also the advantage of being adaptable for use with conventional lagoons.

In the oxidation ditch system which utilizes rotor aeration, a cylindrical rotor is used to aerate the mixed liquor in the ditch and to propel it around the circuit at the same time. Sedimentation and withdrawal of the effluent may be accomplished in several ways, the simplest is by stopping the rotor periodically and allowing the sludge to settle and draining off the supernatant. These systems were developed in Holland and have been operated in communities with 100 to 5,000 population. In conjunction with the oxidation ditch, a secondary clarifier may be provided to permit continuous operation of the rotors.

Consideration has been given to an oxidation ditch for one small community in Ontario and also for a large milk plant although neither of these installations has been constructed to date.

TERTIARY TREATMENT

More consideration has been given to the need for tertiary treatment because of low stream flows and extensive use of receiving streams in the larger urban communities. Effluent sand filters are possibly the most common form of tertiary treatment and under-drained tile beds have been used in some of the smaller installations. Presently, experimental work is being planned utilizing an aerated lagoon to treat effluent from a modern activated sludge plant. Spray irrigation of sewage treatment plant effluent has also been considered in one municipality as a method of tertiary treatment.

PACKAGE PLANTS

There has been a trend towards the use of small, compact, prefabricated sewage treatment plants for shopping centres, motels, hospitals, schools and small subdivisions. Numerous suppliers of package plants are available but by far the most widely used and accepted method is the total oxidation or extended aeration process. Some manufacturers have developed package plants on the biosorption or contact stabilization process as well as contact aeration. Also, one manufacturer supplies a package biofiltration plant.

These plants may be constructed from either steel or concrete and in some of the small package plants, the entire treatment works is pre-assembled at the factory and delivered to the site by truck. The larger installations are field-erected. These plants are designed to handle sewage flows from 500 GPD to 500,000 GPD.

Generally, the total oxidation type plants provide high B.O.D. removals although suspended solids reductions are quite variable. There are a number of operating difficulties associated with these plants which require regular inspection and maintenance.

HYDRAULICS OF PUMPING SYSTEMS

by

A. B. Redekopp

Assistant District Engineer - OWRC

An Address To
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HYDROSTATICS

Fundamental Concepts

Water at rest exerts a pressure in proportion to its depth; the greater the depth, the greater the pressure. In other words, the pressure at the bottom of the column of water is proportional to the height of water in the column. A clear distinction should be made between force and pressure. Pressure is defined as a force per unit area, with the units of pressure usually expressed either as lb/sq. in. (psi), feet of water, or inches of mercury.

In quantitative terms this relationship may be expressed:

$$p = wh$$

where p = pressure at the bottom of a body of water of depth h, generally in lbs. per sq. in. (psi)

h = depth of water, generally in feet

w = specific weight of water, generally 62.4 lbs. per cu. ft. However, specific weight varies with temperature, having a value of 62.42 lbs. per cu. ft. at 32°F. and 59.83 lbs per cu. ft. at 212°F.

This equation indicates that pressure at a point in a liquid of given weight is dependent solely upon the height of the liquid above the point, allowing this vertical height or head to be used as an indication of pressure. The expression "$\frac{p}{w}$" derived from the above equation is called the pressure head. It expresses the depth in feet of the liquid of unit weight "w" required to produce a pressure "p".

Example: Determine the pressure at the bottom of a body of water which is 10 ft. in depth

$p = wh = 62.4 \text{ lbs./ft.}^3 \times 10\text{ft.} = 624 \text{ lbs/ft}^2$

$\frac{624}{144} = 4.33 \text{ lbs/in.}^2 \text{(psi)}$

Relationship of Atmospheric, Gauge, and Absolute Pressures

Figure #1 shows diagrammatically the relationship of atmospheric, gauge and absolute pressures. It is important to understand these relationships as all problems of water pressure fall within this overall frame work. Generally, when water pressure is expressed in psi the reference is to gauge pressure (pressure above atmospheric pressure). In order to express the gauge pressure in terms of absolute pressure, it is necessary to add the atmospheric pressure (approximately 14.7 psi) to the gauge pressure.

Refering again to Figure #1 it is seen that a negative gauge pressure, or a vacuum, is measured below atmospheric pressure, with a maximum theoretical limit of zero absolute pressure. In order to express a negative gauge pressure, or a vacuum in terms of absolute pressure, it is necessary to subtract the negative gauge pressure from atmospheric pressure.

As previously indicated, pressure can be expressed either in terms of lbs. per sq. in. (psi), feet of water, or inches of mercury. An easy way to remember the inter-relationships is in terms of expressions for standard atmospheric pressure.

Standard atmospheric pressure - 14.70 psi
- 33.9 ft. of water
- 29.92 inches of mercury

In order to convert lbs. per sq. in. (psi) to feet of water it would be necessary to multiply the psi by 33.9 over 14.7 or 2.31 lb. per sq. in. (psi)

1 psi = 2.31 ft. of water
1 ft. of water = 0.434 psi

Similarly constants can be developed to convert lbs. per sq. in (psi) or feet of water to inches of mercury.

Example: A mechanical pressure gauge reads 10 inches of mercury vacuum when the atmospheric pressure reads 14.7 psi. What is the absolute pressure?

$$\frac{10 \text{ inches of mercury} \times 14.7 \text{ psi}}{29.92 \text{ inches of mercury}} = 4.91 \text{ psi}$$

absolute pressure = 14.7-4.91 = 9.79 psi

INTERPRETATION OF GAUGE READINGS

In finding pressures at any section under consideration, care must be taken in interpreting gauge readings. The gauge shows the pressure in the spring tube inside it and this may be the same as or different from that in the pipe at the point where the gauge is connected. To illustrate this, imagine a standpipe filled to a depth of 60 feet with water. If a gauge connection is made in the side of the standpipe half way up and a gauge attached so that its centre is opposite the connection, such a gauge will read 30 feet of water. However, if the gauge is located 10 feet below the connection then the gauge will read 40 feet of water. Similarly if the gauge is located 10 feet above the connection the gauge will read only 20 feet of water.

Where the pressure is greater than atmospheric it is necessary to correct the gauge reading by adding the vertical distance in feet that the gauge is above the point of pipe connection, or if the gauge is below the connection, this vertical distance must be subtracted.

A gauge connected directly to a suction pipe showing a vacuum of 20 feet of water would continue to read 20 feet if it were lowered 5 feet by using long piping free of water between the connection and the gauge. If, however, the gauge piping became filled with water then lowering the gauge 5 feet would raise the pressure, indicating a vacuum of only 15 feet. For vacuum gauges it is good practice to blow all the water out of the gauge connection piping by opening for few seconds a petcock connected to such piping close to the gauge.

Unless attention is paid to these points quite serious misunderstanding of the gauge readings may result. Wherever possible, the gauge connection should be short and both air and water tight. It is best to attach the gauge on the same level as the connection.

FLOW OF LIQUIDS

Continuity Principle (based on conservation of mass)

In a continuous pipe system with pipes of varying diameter, the rate of flow (Q) expressed as cubic feet per second (cfs) or gallons per minute (gpm) is the same in each section of pipe. The rate of flow (Q) is equal to the velocity times the cross sectional area of the pipe which when expressed in equation form is:

$$Q = A \times V$$

Where Q = rate of flow in cfs or gpm
A = cross sectional area of pipe, generally in sq. ft.
V = average velocity in pipe, generally in feet per second

When the pipe diameter is changed the cross sectional area (A) changes. This in turn affects the value of the average velocity (V) in order to maintain the equality $Q = AV$. If for example, the diameter decreases, the velocity must increase. This concept is illustrated diagrammatically in Figure #2, where it is noted that the diameter d_2 is less than diameter d_1.

Example: For the illustration of Figure #2 what is the relationship of velocity V_2 to velocity V_1, if $d_2 = \frac{1}{2}d_1$

$$Q = AxV \qquad A_1 V_1 = A_2 V_2 \text{ or } V_2 = \frac{A_1 V_1}{A_2}$$

$$\text{but } A_1 = \frac{d^2{}_1}{4} \quad \text{and } A_2 = \frac{\left(\frac{d_1}{(2)}\right)^2}{4}$$

$$\text{Consequently } V_2 = 4V_1$$

An important point to be made here is, that where the diameter is cut in half the velocity in the reduced section is increased four times. Also, the net effect of reducing the pipe diameter by one half would be to increase the pressure loss due to friction approximately 32 times for any given length of pipe.

Energy Relationships for Flow in Pipes

Important to the understanding of the flow of liquids in lines is a knowledge of the energy relationships involved. Where the fluid is a liquid the total energy at point 2 in a pipeline is equal to the sum of the pressure head the velocity head and the potential head. This concept is known as the Bernoulli Principle and is illustrated in Figure #3. It should be noted that the energy loss due to pipe friction and fittings, which is generally dissipated as heat loss and is, therefore, not available as usable energy at point 2 is subtracted from the pressure head. Also it is seen that an increase in the potential head at point 2 further reduces the pressure head at this point. If glass tubes were taped into the pipe wall at point 1 and point 2 the water would rise to a height corresponding to the pressure at the point expressed as feet of water. The hydraulic gradient is the line joining the tops of these water columns.

The Bernoulli Principle is expressed as follows:

$$\frac{P_1}{62.4} + \frac{V_1^2}{64.4} + Z_1 = \frac{P_2}{62.4} + \frac{V_2^2}{64.4} + Z_2 + H_L$$

where $\frac{P_1}{62.4}$ = pressure head expressed in ft. of water (P should be in lbs./ft.^{2})

$\frac{V^2}{64.4}$ = velocity head expressed in ft. of water (V should be in ft./sec.)

Z_1 = elevation head expressed in ft.

H_L = friction loss in pipe and fittings expressed in ft. of water.

This is the cardinal principle in hydraulics. For a further discussion on this subject reference should be made to a standard hydraulics text book.

Friction and Resistance

Suppose that gauges are attached at some distance apart on a horizontal pipe of uniform size with water flowing through the pipe every practical main knows that the downstream gauge always reads less because of the known effects of friction of the water on the walls of the pipe. The difference in pressure increases as the distance between the gauge connections increases, as the flow increases and as the pipe gets older.

Pipe friction causes the loss of considerable non-recoverable energy in all pipe lines. This friction results from the movement of liquid particles past one another and from turbulence caused by projections from the pipe interior.

Experiments show that the loss of head due to friction varies almost exactly in proportion to the velocity head. Several formulae have been developed to calculate the friction loss. The Hazen - Williams formula is probably the most widely used. Fortunately, tables and charts have been prepared and may be obtained from any pipe manufacturer.

PUMPS

Definitions

A pump may be broadly defined as a mechanical device capable of transmitting liquids and gases through pressure conduits from one elevation or pressure to a higher elevation or pressure. To accomplish this transfer, energy must be imparted to the fluid by the pump. In hydraulic calculations, energy is known as head. Head may be defined as a value which describes the amount of energy contained in each pound of fluid under a particular condition. It is generally expressed in units of foot-pounds per pound; these units are usually reduced to feet. Consequently, head is generally expressed in feet. The different heads involved in pump work are described below.

Friction Head - is the pressure expressed in feet of water

needed to overcome the resistance to the flow in the pipe and fittings.

Suction Lift - exists when the source of supply is below the central line of the pump.

Suction Head - exists when the source of supply is above the central line of the pump.

Static Suction Lift - is the vertical distance from the central line of the pump down to the free level of the liquid source.

Static Discharge Head - is the vertical elevation from the central line of the pump to the point of free discharge.

Velocity Head - is the head needed to accelerate the liquid. Knowing the velocity of the liquid from the continuity equation $Q = AV$, the velocity head can be calculated by the simple formula $\frac{V^2}{64.4}$ Although the velocity head is a factor in figuring the dynamic heads, the value is usually small and in most cases negligible.

Dynamic Suction Lift - Includes static suction lift plus friction head plus velocity head.

Dynamic Suction Head - includes static discharge head minus friction head minus velocity head.

Total Dynamic Discharge Head - includes static discharge head plus friction head plus velocity head.

Total Dynamic Head - includes the dynamic discharge head plus dynamic suction lift or minus dynamic suction head. The total dynamic head is the basis for selection of a pump.

Suction Limitations - (any pump)

The importance of keeping within the suction limitations of any pump (centrifugal, rotary, piston,) cannot be emphasized too greatly. A pump, by creating a vacuum at the suction (Impeller eye on a centrigual) utilizes atmospheric pressure to push the liquid into a pump. Because of this, the suction lift is limited theoretically to 33.9 feet of water. Internal pump losses reduce this limitations even more. The dynamic suction lift should be calculated carefully at the required capacity to make sure that it is within the pumps capabilities. The maximum practical dynamic suction lift is 25 ft. of water , this indicates that the static suction lift should not exceed 15 feet. In order to reduce the friction head in the suction pipe it will usually be found that the suction pipe is larger than the discharge pipe. Always keep the pump as close to the liquid source as possible.

Pump Selection

Much may be said on this subject and it would require several lectures to consider the technical aspects necessary to make a wise selection. The most important characteristics of pumps used in sewage should be reliability and freedom from clogging, as even a short interruption of service may create a serious situation. An interruption of service may cause sewage to back up into a home, flood the lift station, or some other undesirable occurrence. Other important characteristics of a sewage pump should be resistance to wear, ease of unclogging, and efficiency of operation.

A factor which is fairly important in sewage pumping selection is the capacity of the pump in relation to size. Pump sizes are given as the diameter of the discharge connection. It is impracticable in most cases to make sewage pumps smaller than 4 inches, because the sewage pump should be able to pass a $2\frac{1}{2}$ inch diameter solid. Most authorities agree that about a 2 ft./sec. velocity in the suction and discharge lines is necessary to ensure that no accumulation of solids will occur in the line. In order that this velocity may be attained, it is necessary to have about 75 gpm flowing through a 4 inch diameter pipe. For this reason, a capacity of less that 75 gpm for a 4 inch pump should not be specified unless some unusual conditions exist.

Pump Impeller Trimming

The impeller of a pump may be trimmed down to reduce the capacity of the pump. In order to establish the new pump characteristics the following relationships may be employed.

Capacity proportional to speed or impeller diameter
Head proportional to $(\text{speed})^2$ or $(\text{impeller diameter})^2$
Power proportional to $(\text{speed})^3$ or $(\text{impeller diameter})^3$

Example: The impeller of a 12 inch diameter pump rated at 4,000 gpm and a head of 140 feet, requiring 163 bhp was trimmed to $11\frac{1}{2}$ inches. Calculate the new capacity, the new head and the power requirements.

$$\frac{\text{New capacity}}{\text{Old capacity}} = \frac{\text{New diameter}}{\text{Old diameter}} \qquad \frac{X}{4000} = \frac{11.5}{12}$$

$$\therefore \text{New capacity} = X = 4000 \times \frac{11.5}{12} = \underline{3830 \text{ gpm}}$$

$$\frac{\text{New Head}}{\text{Old Head}} = \frac{(\text{New diameter})^2}{(\text{Old diameter})^2} \qquad \frac{Y}{140} = \frac{(11.5)^2}{(12)^2}$$

$$\therefore \text{New head } Y = 140 \times \frac{(11.5)^2}{(12)^2} = 140 \times 0.96 = 128 \text{ feet}$$

$$\frac{\text{New power}}{\text{Old power}} = \frac{(\text{New diameter})^3}{(\text{Old})\text{diameter})^3} \qquad \frac{Z}{163} = \frac{(11.5)^3}{(12)^3}$$

$$\therefore \text{ New Power} = Z = 163 \times \frac{(11.5)^3}{(12)^3} = 163 \times 0.877 = \underline{143 \text{ Hp}}$$

Variable Speed Sewage Pump Drives

By using variable speed drives, a large range of flow rates can be matched by exact pumping rates. This means a minimum number of pump units per pumping station. For instance, two variable speed pumps can, with considerable precision match many more flow rates than three or four constant speed pumps. This reduces the size of the station (dry pit, wet well & piping)

The reduction in size of the wet well and the continuous flow of sewage reduces the settling of solids in the wet well and the discharge mains.

Frequently a lift station is to be located in an area of large potential growth in population or industry. By using variable speed pumps it is possible to provide economical operation with the initial installation and to provide for the expected growth. For the early years the pumps will operate at low speeds and as the area served grows, the pump speed will automatically increase to meet increased demand. A single pump installation can by designed to meet a growth factor for a number of years.

The frequent on-off operation (or infrequent in certain hours of the day) of constant speed pumps is undesirable for influent and intermediate pumping station at sewage treatment plants. Variable speed pumping provides an even flow of sewage into and through the plant, thereby allowing biological and other processes to operate continuously and more effectively.

Many existing lift stations could be modified the employ variable speed drives with a nominal investment and great advantage to the user.

FIGURE 1

RELATIONSHIP OF ATMOSPHERIC, GAGE AND ABSOLUTE PRESSURES

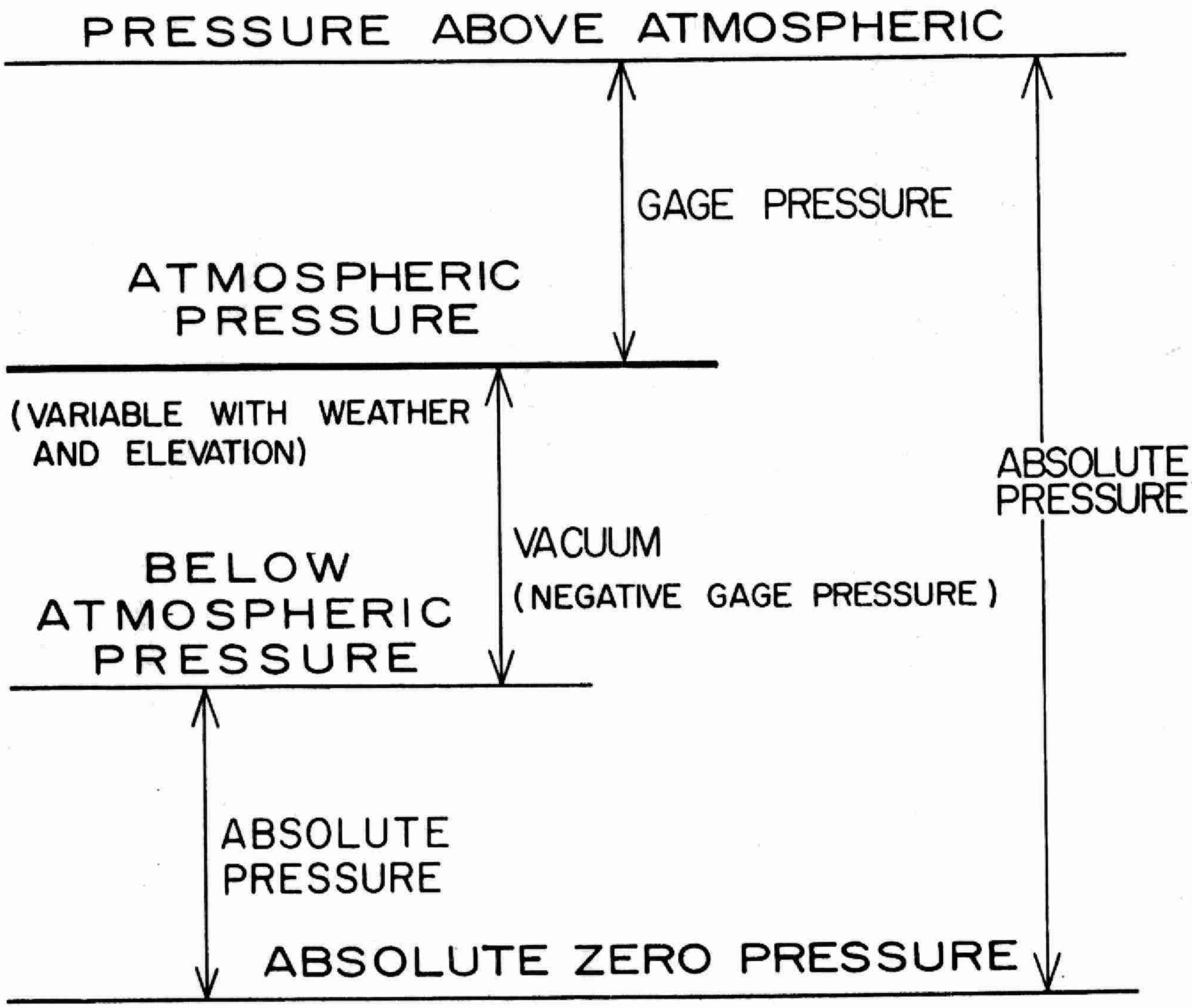
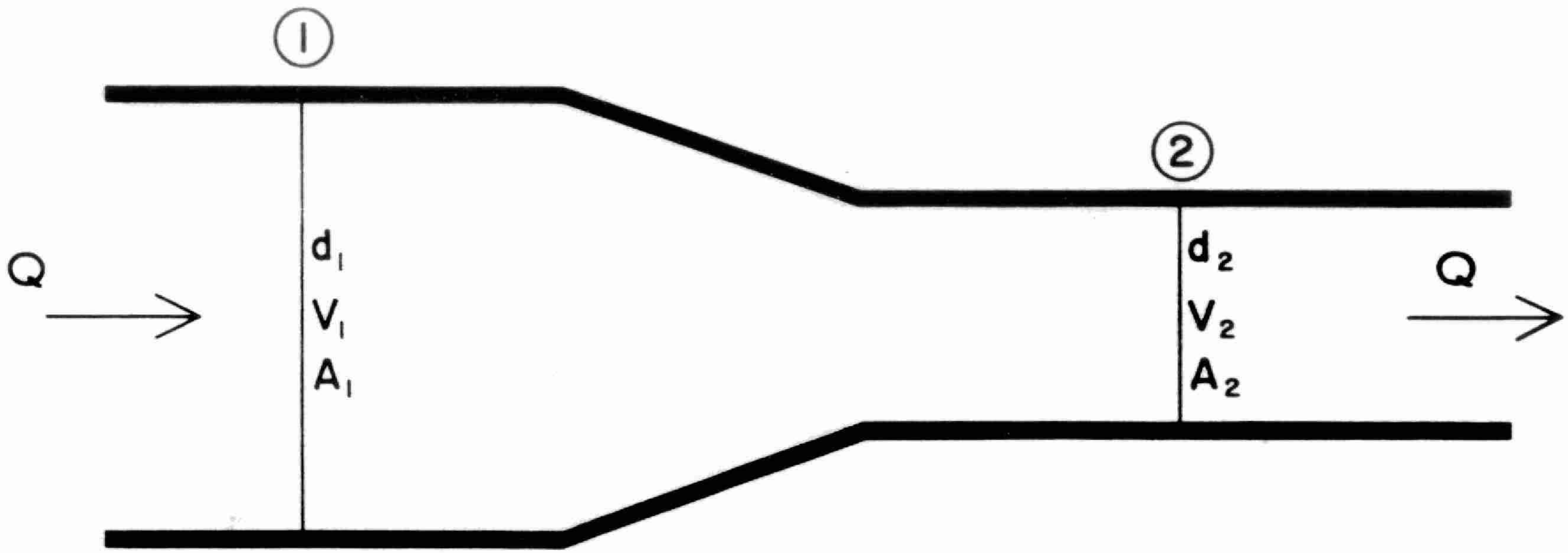


FIGURE 2

CONTINUITY PRINCIPLE



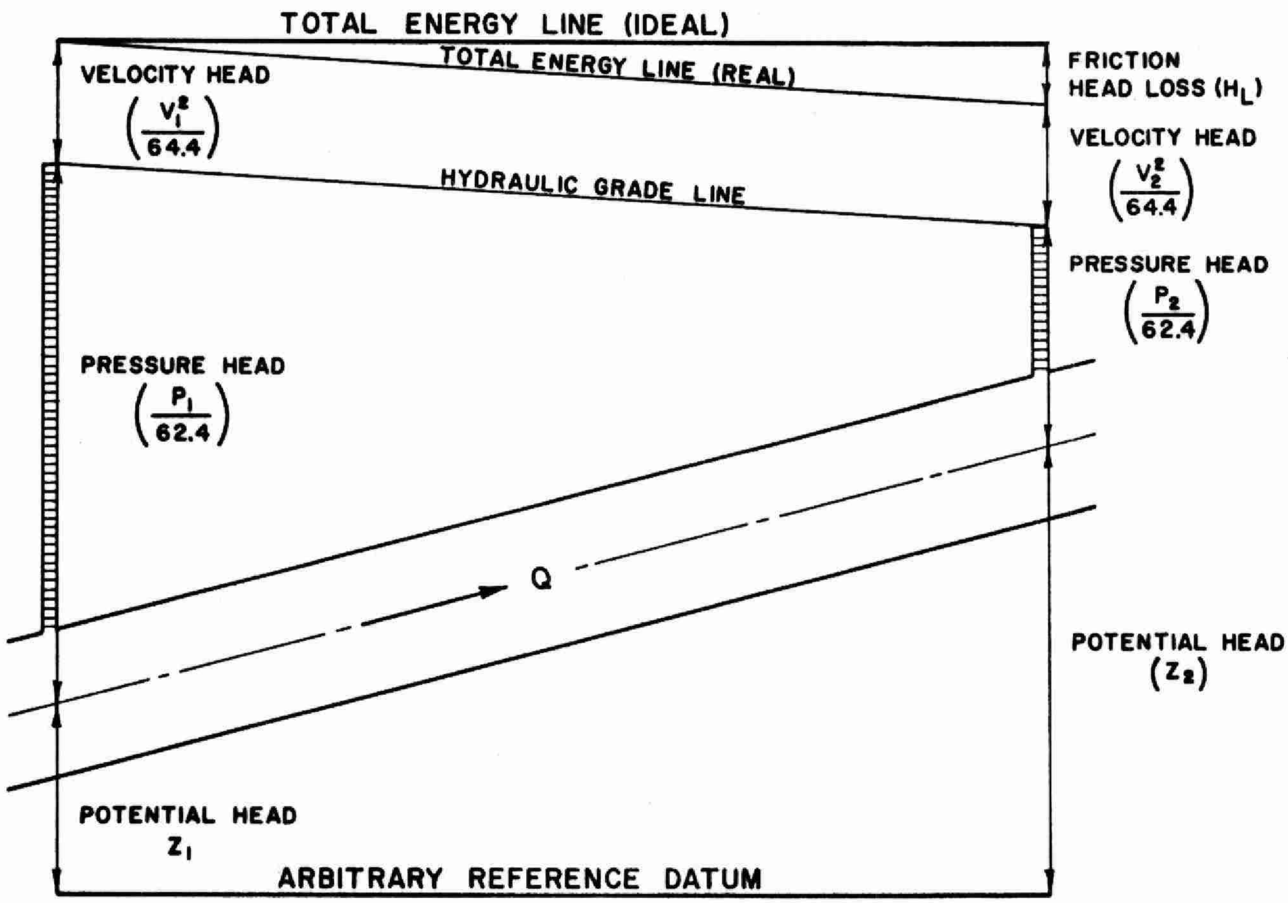
$$Q = A_1 V_1 = A_2 V_2$$

$$V_2 = \frac{A_1 V_1}{A_2}$$

FIGURE 3

ENERGY RELATIONSHIPS FOR FLOW IN PIPES

(BERNOULLI PRINCIPLE)



(POTENTIAL HEAD)$_1$ + (PRESSURE HEAD)$_1$ + (VELOCITY HEAD)$_1$ =

(POTENTIAL HEAD)$_2$ + (PRESSURE HEAD)$_2$ + (VELOCITY HEAD)$_2$ + FRICTION HEAD LOSS.

HYDRAULICS OF OPEN-CHANNEL FLOW

by

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INTRODUCTION - DEFINITION OF TERMS

Open Channel

The term open channel means any conduit in which water flows with a free surface. All rivers, canals, flumes, and other uncovered conduits are thus classed as open channels as are also such closed conduits as underground drains, sewers, and tunnels when flowing partially full.

One important difference between conduits carrying water under pressure and those in which flow occurs with a free surface is that in the former the stream is confined and its cross section is fixed, while in the latter the stream is unconfined and therefore free to expand or contract. Many interesting and important phenomena in open channel flow, made possible by the freedom of the stream to change its section, cannot occur in closed conduits flowing full.

Steady and Unsteady Flow

If at any cross section of a stream the discharge is constant, the flow is said to be steady, and if the discharge changes during successive time intervals the flow is unsteady. In order that steady flow may be maintained within any reach of a channel, it is necessary that the quantity of water entering and leaving the reach remain constant. The flow in local sewers is usually unsteady but in order to simplify hydraulic design steady flow conditions are assumed.

Uniform and Non-Uniform Flow

Uniform flow occurs when the velocity and depth are the same from point to point along the conduit. Uniform flow is possible only in a channel of constant cross section.

If at successive cross sections the velocities are not the same the flow is non-uniform. Non-uniform flow occurs in all channels where there is either accelerated flow or backwater.

UNIFORM FLOW

Energy and Head Considerations

The open channel illustrated in Fig. la carries water at a depth D. The energy possessed by the water is made up of two parts, the kinetic or motive energy and the potential or latent energy. The potential energy results from the position of the water and also from the pressure which it produces.

The total energy per pound of water at any cross section referred to the horizontal datum plane is

$$H = Z + D + V^2/2g.$$

The energy in a cross section referred to the bottom of the channel is termed the specific energy

$$E = D + V^2/2g.$$

The line representing energy head of a stream which is every where $V^2/2g$ above the water surface is termed the energy gradient.

As illustrated in Fig. 1b for pressure conduits the pressure (P/W) in the pipe may vary along the pipe and depend upon energy losses and the conditions imposed under the ends of the line. Open flow, however, is characterized by a constant pressure, usually atmospheric, existing on the entire surface of the flowing liquid. This leads to the conclusion that for open channel flow the hydraulic grade line lies in the liquid surface.

For uniform flow in an open channel, the slope of the energy grade line and the hydraulic grade line will be the same as the slope of the invert, and the depth of flow will adjust itself to produce a velocity commensurate with the friction losses in the channel.

Losses of Head

Losses of head in open channels are analogous to corresponding losses in pipes. There is a continuous and gradual loss of head termed loss of head due to friction which occurs throughout the channel. The friction head must be estimated by means of an empirical formula based on observed valves of friction loss. There is also an additional loss termed minor

loss wherever abrupt changes in velocity occur and wherever the direction of flow changes.

The only factor that affects the difference in elevation of the energy grade line between two cross sections is the friction loss. Therefore the energy grade line is used as the base for hydraulic computations rather than the invert of the conduit.

Energy Loss Formulae for Open Channels

One of the earliest expressions for energy loss was developed by Chezy in 1775.

$$V = C\sqrt{RS}$$

where C = Chezy coefficient
R = hydraulic radius
= area of flow cross section divided by the wetted perimeter.
S = average loss of head due to friction per foot
= hf/L where hf is the total loss of head in a reach of length L. It is therefore the slope of the energy gradient, and if flow is uniform, it is also the slope of water surface and the grade of the channel.

Many experiments have been performed to determine the magnitude of the Chezy coefficient, "C", and its dependence on other variables. For the turbulent flow of water in open channels the Chezy coefficient is dependent upon the roughness, hydraulic radius, and slope of the channel.

Kutter's formula for "C" was derived in 1869 from experimental results obtained from hydraulic tests on artificial and natural channels of all descriptions, ranging in size from small laboratory channels up to large rivers.

The formula is commonly used in the solution of problems involving open-channel flow in sewers. Graphs for its use are available in most texts on sewer design.

Manning's formula presented in 1890 is often used for the solution of problems involving flow in open channels.

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

where n = coefficient of roughness

The expression 1.486/n was designed to make valves of "n" correspond to the valves of "n" in the more cumbersome Kutter's formula.

Convenient tables, graphs, and slide rules are available for the solution of the Manning's Formula.

Some valves for "n" are given in Tables 1 and 2 below.

Best Hydraulic Cross Section

Considered purely from the standpoint of the hydraulics of a channel, the most efficient cross section is the one which, with a given slope and area, has the maximum capacity. It can be seen from an examination of the Manning's formula that this cross section is the one having the maximum hydraulic radius. The semicircle has the highest hydraulic efficiency of all open channel sections. The rectangular section of highest efficiency has a depth equal to one-half the width.

TABLE 1

Suggested Values of "n" for Various Pipe

Kind of Pipe	From	To
Cement-lined Cast Iron	0.012	0.015
Concrete pipe	0.015	0.035
Asbestos Cement pipe	0.012	0.015
Vitrified sewer pipe	0.012	0.015
Corrugated steel pipe		
Uncoated, ½ inch corrugations	0.024	0.026
Asphalt coated and 25% paved	0.021	0.023
Smooth asphaltic lining	0.012	0.015

TABLE 2

Miscellaneous Values of the Roughness Coefficient, "n"

Smooth cement, planed timber	0.010
Rough timber, canvas	0.012
Good ashlar masonry or brickwork	0.013
Rubble masonry	0.017
Firm gravel	0.020
Canals and Rivers in good condition	0.025
Canals and Rivers in bad condition	0.035

Variation of Velocity & Rate of Flow with Depth in Sewers

Frequently in sewer problems closed conduits do not flow full. Open-channel flow in closed conduits possesses certain special features because of the convergence of the side walls at the top of the conduit. For sewer design the variation of velocity with depth should be determined since certain velocities must be maintained in order to transport suspended solid matter.

An analysis of the controlling variables carried out by using Manning's formula results in a useful diagram, Fig. 2, in which the velocities and rates of flow are both unity when the conduit flows full. The diagram gives, for any depth, the ratio of rate of flow and velocity at that depth to those when the conduit flows full. With rate of flow and velocity for the full conduit easily obtainable from Manning's formula, rates of flow and velocities for a partially full conduit can be readily determined with the aid of the diagram.

The depths at which maximum flow and maximum velocity occur are seen to be 0.96D and 0.95D respectively, for the conduit of circular cross section.

This hydraulics-elements graph was based on the work of Wilcox and of Yardell and Woodward which showed that "n" increases as the depth of flow in the conduit decreases.

Earlier hydraulic elements graphs based on the erroneous assumption that the friction coefficient "n" does not vary with the depth gave depths for maximum flow and maximum velocity of 0.94D and 9.80D respectively.

Example:

An 8"Ø public sewer serving 20 lots has been installed on a given street at a minimum slope of 4 feet per 1000 feet which would produce a velocity of 2 fps if flowing full.

The maximum flow to be carried is about

$$\frac{20\text{ (lots)} \times 4\text{ (persons/lot)} \times 400}{540{,}000\text{ gals./cfs}} \text{ (gallons per capita per day)} = 0.03 \text{ cfs.}$$

What is the actual velocity of flow in the sewer?

The 8"Ø sewer when flowing full can carry 0.70 cfs, using n = 0.13 in the Manning's formula.

The ratio q/Q full = $\frac{0.03}{0.70}$ = 0.043

From the graph d/D = 0.15

and v/V full = 0.4

Therefore the actual velocity of flow under maximum conditions is 2.0 x .4 = 0.8 fps which will result in settling out of sewage solids.

Velocity for Equal Cleansing in Sewers

The mean velocity in a sewer may be adequate for self-cleansing at one particular rate of flow but not at lower rates of flow. The critical velocity for scour not only varies with discharge but also with the change in friction factor with the depth of flow.

Hydraulic elements, based on the assumption that sewers flowing at partial depth will remain as self-cleaning as sewers flowing full, if the intensity of the tractive force on sewage deposits remains the same at all depths of flow, are given in Fig. 3.

The graph shows that no change in slope is required when the sewer flows more than half full but that the slope must be doubled when the flow drops to 0.2 full and quadrupled at 0.1 depth.

Example:

From the previous example q_s/Q_{full} = 0.043, and d/D = 0.15

From Fig. 4 the required scouring velocity V is 2.0 x 0.68 = 1.36 fps which is more than the actual velocity of 0.8 fps.

In order to prevent settling out the slope should be increased from 4 ft. per 1000 ft. to 4.0 x 2.7 = 10.8 ft. per 1000 ft.

NON-UNIFORM FLOW

Non-uniform flow in open channels may be rapidly varied where sudden changes in cross-sectional area occur or gradually varied where gradual changes occur in cross-sectional area.

Rapidly Varied Flow

The principles of rapidly varied flow may be best derived by considering the specific energy of the water flowing in an open channel.

If a uniform velocity distribution is assumed in a rectangular channel, the unit rate of flow, q, through a vertical strip of 1-ft. width will be given by $q = Vd$ which will eliminate the width of channel, b, from subsequent equations. Then q is related to the total rate of flow, Q, by $Q = bq$.

From the foregoing equation for q, $V = q/d$, which may be substituted in the equation for specific energy presented previously giving

$$E = d + \frac{q^2}{2gd^2}$$

A thorough understanding of this equation and its physical meaning may be obtained most easily by assuming q constant and studying the relation of E and d. Plotting this equation gives the "specific energy diagram" of Fig. 4 and introduces some new concepts to open-channel flow.

In the uniform flow of a certain quantity of liquid in open channels of various slopes, it is evident that steep slopes will tend toward high velocities and small depths, and mild slopes will tend toward low velocities and large depths. The slopes thus determine the depths, but the depths in turn determine the specific energy since q is constant.

The specific-energy curve possesses a point of minimum energy at which the depth is termed the "critical depth." For every specific energy, "E", there are two "alternate" depths at which flow may take place, one greater and one less than the critical depth. If the depth of flow is greater than the critical depth, the flow is said to be "tranquil" and the slopes which bring about such flows are designated as "mild" slopes. The flow is said to be "rapid" if its depth is less than the critical depth; rapid flows are caused by "steep" slopes. Uniform flow at the critical depth will occur when the channel has a "critical slope," S_c.

Certain general characteristics of open-channel flow may be deduced from the specific-energy curve. In the region of flow near the critical depth, the depth may change for practically constant specific energy. Physically this means that since many depths may occur for practically the same specific energy content, flow near the critical depth will possess a certain instability which frequently manifests itself by undulations in the liquid surface. It is also evident from

the curve that a loss of specific energy will be accompanied by a reduction in depth in tranquil flow, but in rapid flow an increase in depth is associated with a loss of specific energy.

Critical-Depth Relationships

Since critical depth occurs when specific energy is minimum for a given rate of flow, the equations of critical flow may be obtained by differentiating and equating the result to zero.

$$q^2 = g d_c^2 \qquad \text{or} \qquad q = \sqrt{g} d_c^{3/2}$$

$$\text{and} \quad Q = b \sqrt{g} d_c^{3/2}$$

Critical depth depends only upon the rate of flow in the channel. The equation suggests utilizing critical flow as a means of metering open-channel flow. If critical flow may be caused to exist in a channel its depth may be measured and the rate of flow calculated.

Occurrence of Critical Depth

Critical depth will occur in open-channel flow when a change in channel slope brings about a change from tranquil to rapid flow or from rapid to tranquil flow. In Fig. 5, critical depth occurs where a change from a mild slope to a steep slope causes the flow to pass through the critical depth in its smooth transition from tranquil to rapid flow.

At free outfall from a rectangular channel of mild slope (Fig 6a), the critical depth will occur a short distance upstream from the brink. For very mild slopes the brink depth is a fixed proportion of the critical depth given by

$$\frac{d_o}{d_c} = 0.655 \quad \text{or} \quad d_c = 1.52\ d_o$$

Substituting the above relation gives

$$Q = 1.87\ \sqrt{g} d_o^{3/2}$$

$$= 10.6b\ \ d_o^{3/2}$$

thus allowing rate of flow to be calculated from brink depth.

The Hydraulic Jump

In Fig. 5 an abrupt change from rapid to tranquil flow occurs when the slope is again changed to a mild slope. **The** hydraulic jump may occur in an open channel when water flowing at a high velocity is retarded. The transition is always from a stage less than critical depth to a stage greater than critical depth but less than the higher of the two stages of equal energy.

For rectangular channels, where v_1 and v_2 are the velocities before and after the jump; and D_1 and D_2 are, respectively, depths before and after the jump:

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2v_1^{\,2}D_1}{g} + \frac{D_1^{\,2}}{4}}$$

and

$$D_1 = -\frac{D_2}{2} \quad \sqrt{\frac{2v_2^{\,2}D_2}{g} + \frac{D_2^{\,2}}{4}}$$

GENERAL SEWAGE TREATMENT PLANT HYDRAULICS

Sewage flowing through the various treatment units requires a difference in elevation of the sewage level between the entrance and the outlet to overcome the various hydraulic losses. These head differences will vary with the rate of flow. The successful operation of the plant depends, in a large measure, on the skill of the consulting engineer to provide a good hydraulic design.

Head losses at various points through the treatment works may be classified as follows:

(1) Friction loss through conduits;
(2) Velocity-head losses;
(3) Heads required for discharge over weirs, through orifices and other controlling and measuring devices;
(4) Water-level drops at various points such as free-fall over weirs;
(5) Head allowance for future extensions;
(6) Head allowance for high-water in the receiving waterway.

Hydraulic Computations for Major Elements

The hydraulic computations are started with the high water level of the river or receiving body of water and extended up through the outlet sewer and the plant in reverse direction to the flow of the sewage. The hydraulic profile should be computed for the minimum, average and maximum flow rates in order to determine the proper elevations for the various plant elements, and the most suitable sizes for interconnecting channels and pipes. Low flow conditions tend to produce less than minimum velocities resulting in the settling out of solids which is undesirable.

To give examples of typical computations is beyond the scope of this paper. Although non-uniform flow usually occurs throughout, uniform flow conditions are assumed in order to simplify the calculations. The resulting liquid surface profiles are considered accurate enough for all but the largest treatment plants or special flow distribution problems where model studies may be conducted to verify the theoretical calculations.

A plan view of a typical activated sludge plant of 2.0 M.G.D. capacity is given in Fig. 9. The hydraulic profile for maximum flow conditions is shown for illustration purposes in Fig. 10.

Total Head Requirements

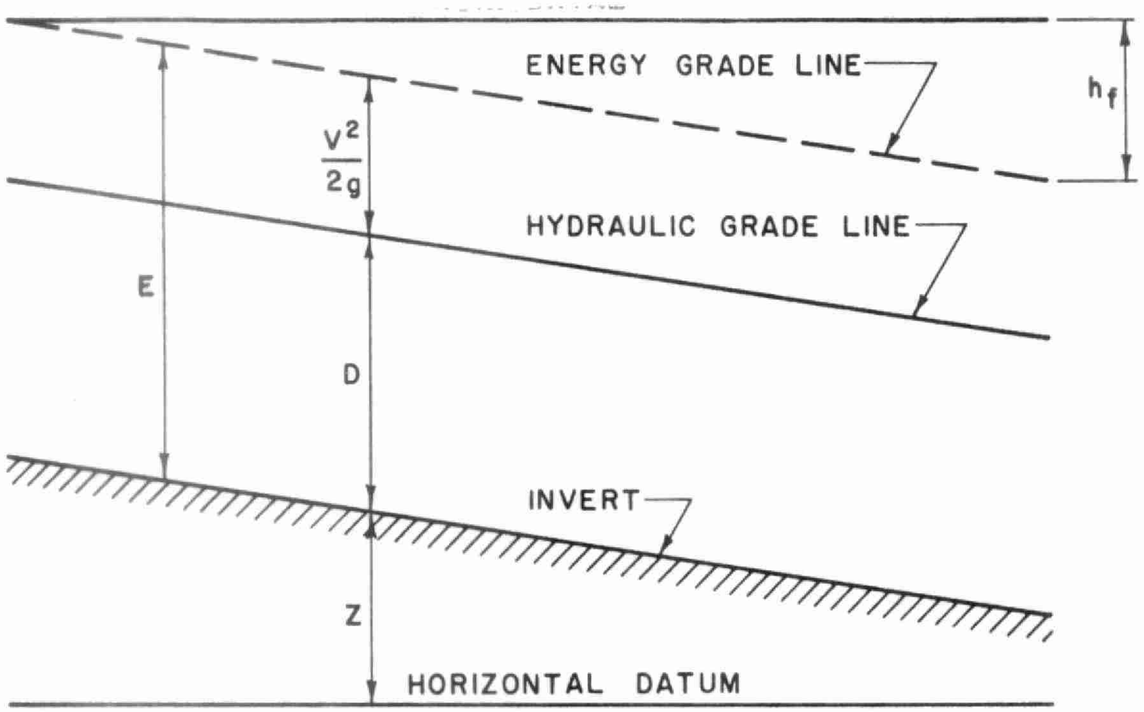
The overall head requirements vary for different types of sewage treatment plants and for different plants of the same type. Some examples are given in Table 3 below:

TABLE 3

Plant and Type		Rated Plant Capacity - U.S. M.G.D. Avg. Flow	Rated Plant Capacity - U.S. M.G.D. Max. Flow	Loss of Head Thru Plant At Avg. Flow	Loss of Head Thru Plant At Max. Flow
Plain Sedimentation					
A		17.3	50.0	1.25	3.07
B			30.0		3.45
C		150.0	570.0	4.34	6.22
Trickling Filters					
D depth of stone	7.5'		8.0		23.67
E	8.0		10.3		22.00
F	8.0	1.45	2.7	17.65	18.04

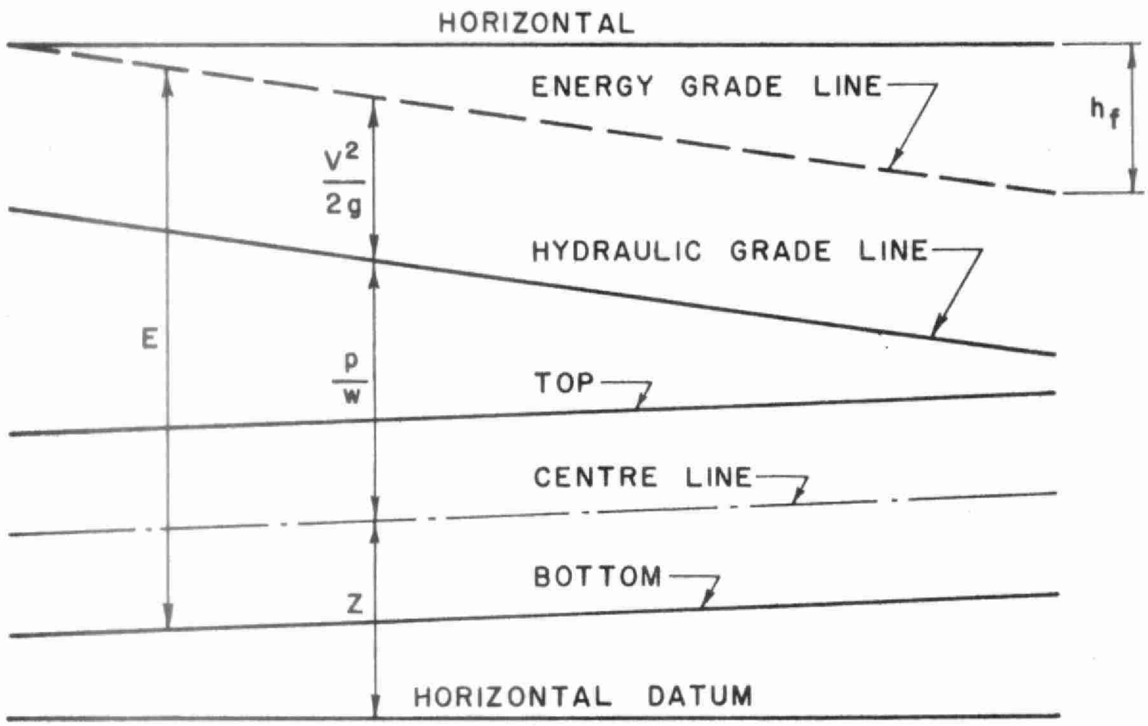
Plant and Type	Rated Plant Capacity - U.S. M.G.D.		Loss of Head Thru Plant	
	Avg. Flow	Max. Flow	At Avg. Flow	At Max. Flow
Activated Sludge				
G		33.75		3.42
H	22.0	50.0	2.50	2.88
I		231.0		4.25
J	175.0	370.0	3.06	5.30

The total head loss for the activated sludge plant of Figs 9 and 10 at maximum flow of 5.9 feet is greater than that indicated above since liberal allowances were made for free-fall over all weirs.



$$H = Z + D + \frac{V^2}{2g} \qquad E = D + \frac{V^2}{2g}$$

(A) OPEN CHANNEL

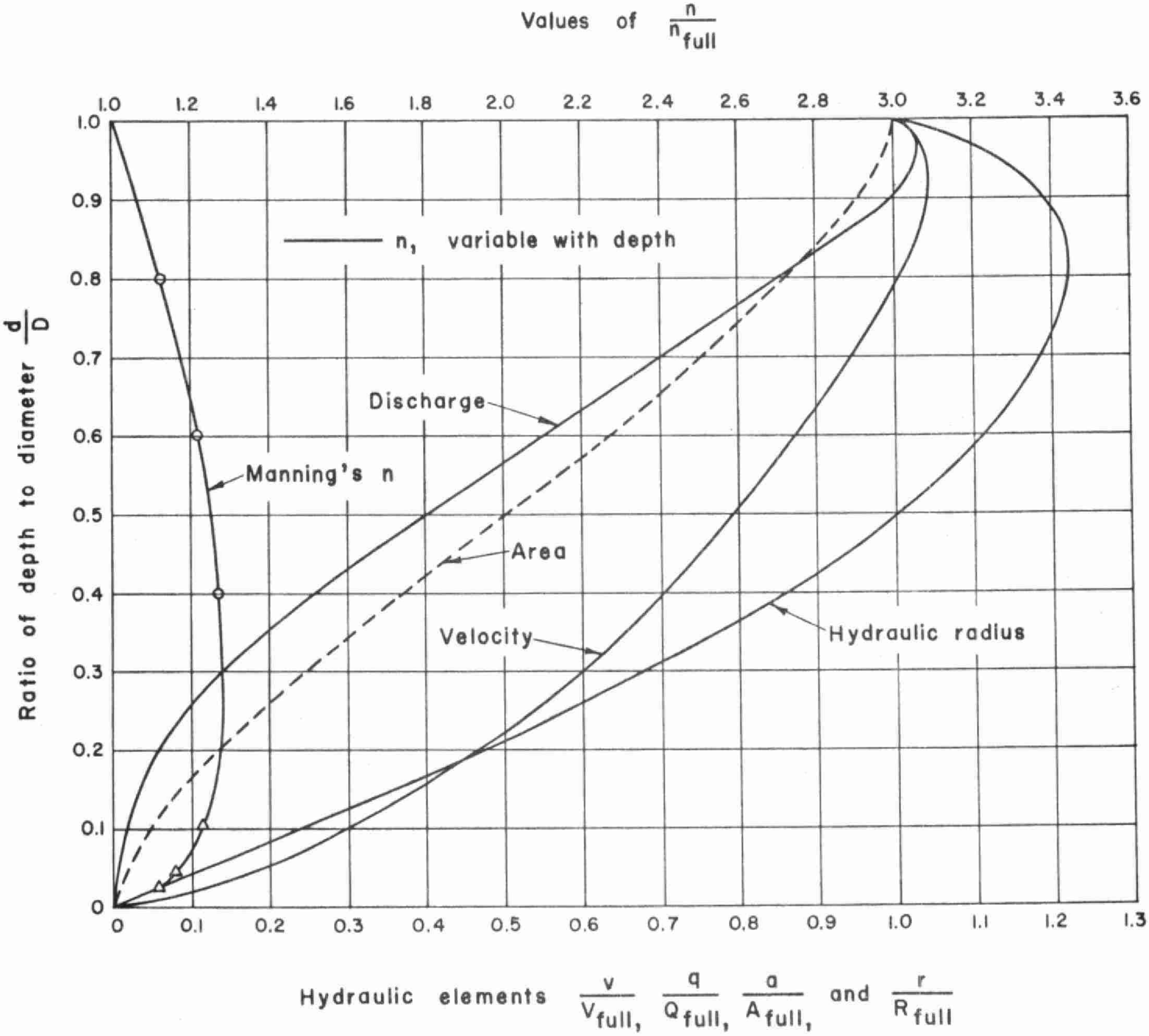


$$H = Z + \frac{p}{w} + \frac{V^2}{2g}$$

(B) PRESSURE CONDUIT

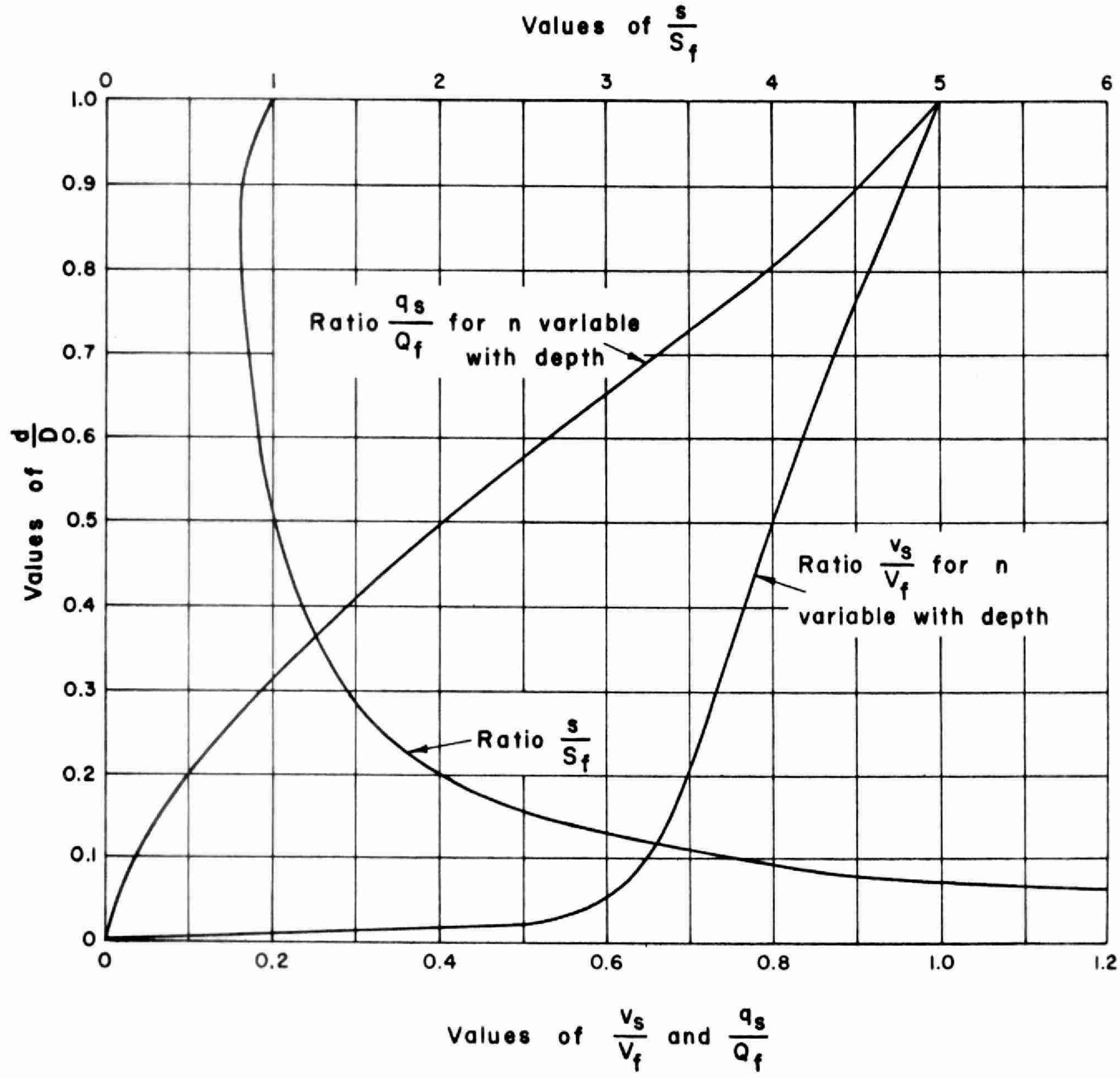
UNIFORM FLOW HYDRAULIC PROFILES

FIG. 1



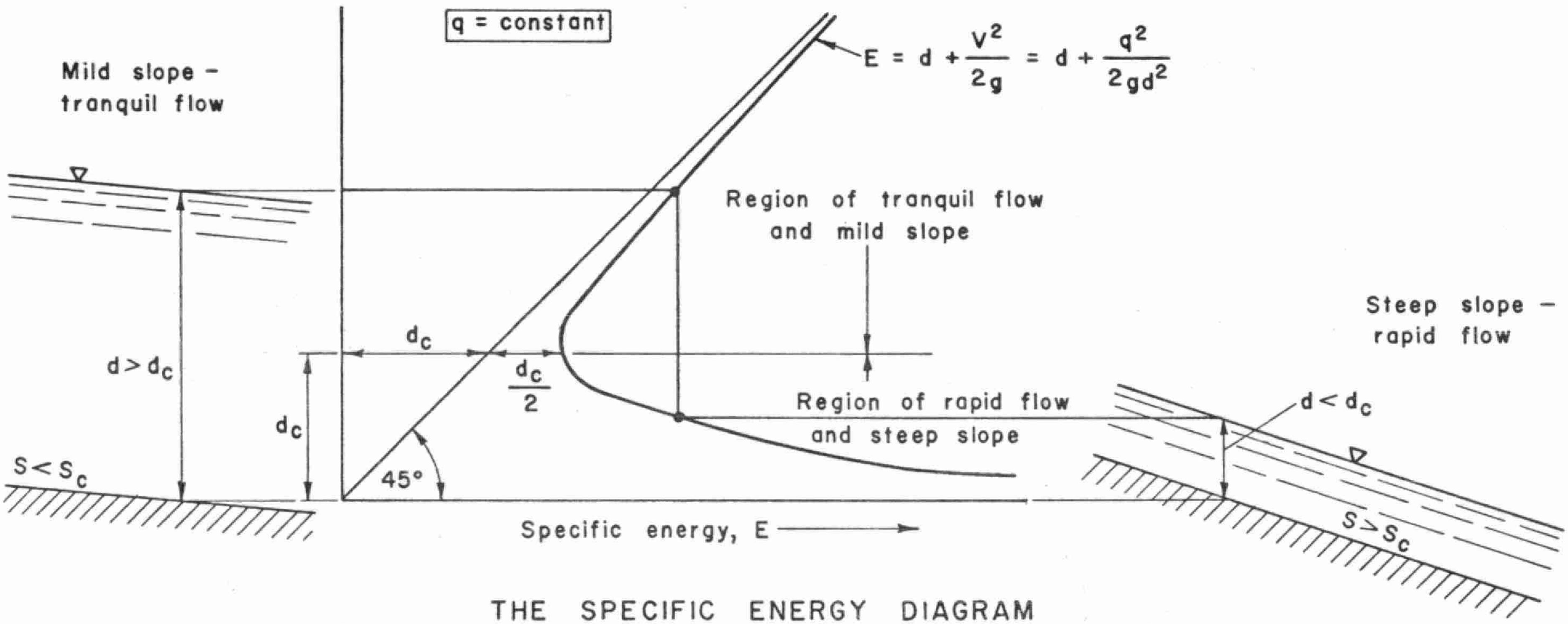
HYDRAULIC — ELEMENTS GRAPH FOR CIRCULAR SEWERS

FIG. 2



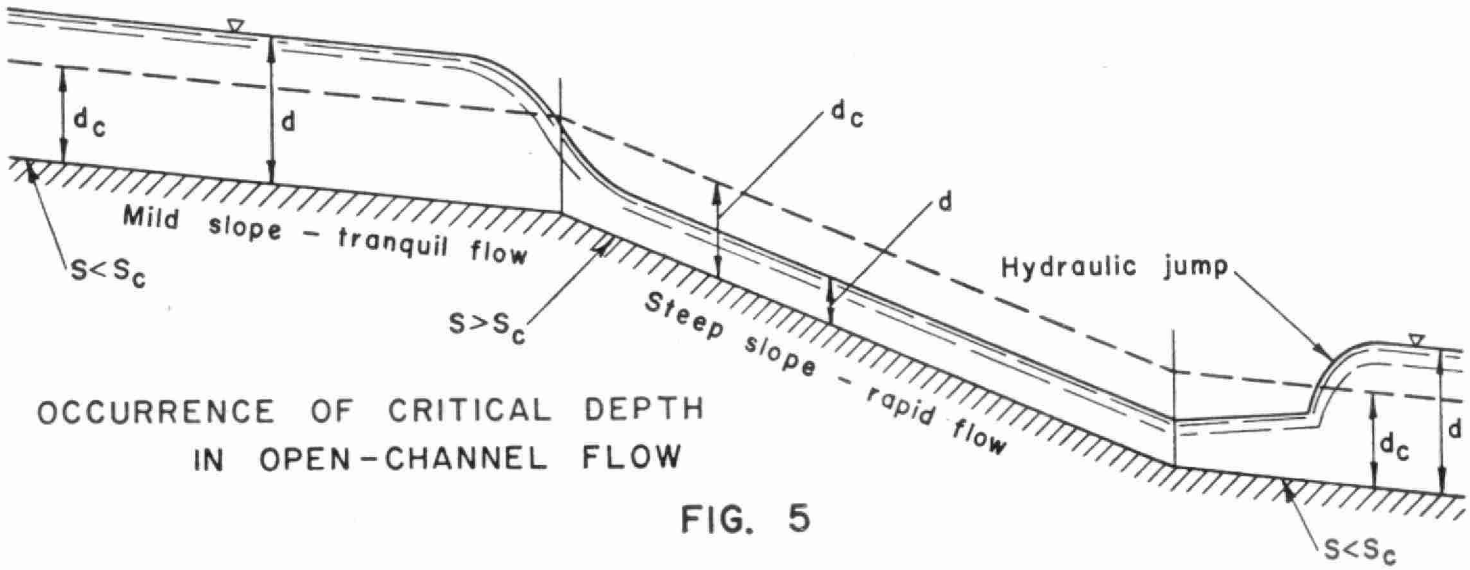
HYDRAULIC ELEMENTS OF CIRCULAR SEWERS THAT POSSESS EQUAL SELF - CLEANSING PROPERTIES AT ALL DEPTHS (SMALL LETTERS APPLY TO PARTLY FILLED CONDUITS)

FIG. 3



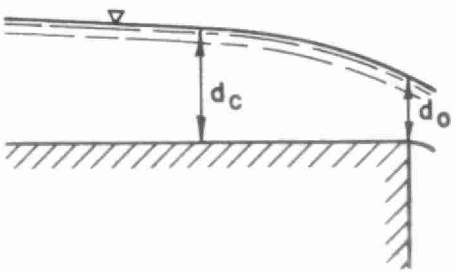
THE SPECIFIC ENERGY DIAGRAM

FIG. 4



OCCURRENCE OF CRITICAL DEPTH
IN OPEN-CHANNEL FLOW

FIG. 5

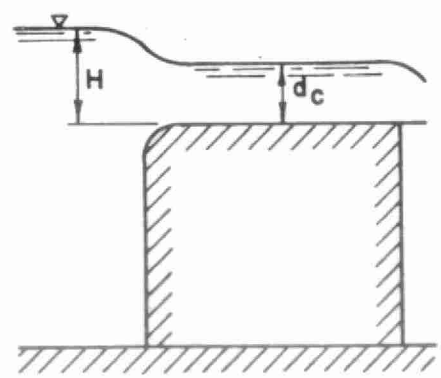


$$Q = 1.87\ b\sqrt{g}\ d_o^{\frac{3}{2}}$$

Where b = width of the channel

FREE OVERFALL

FIG. 6a

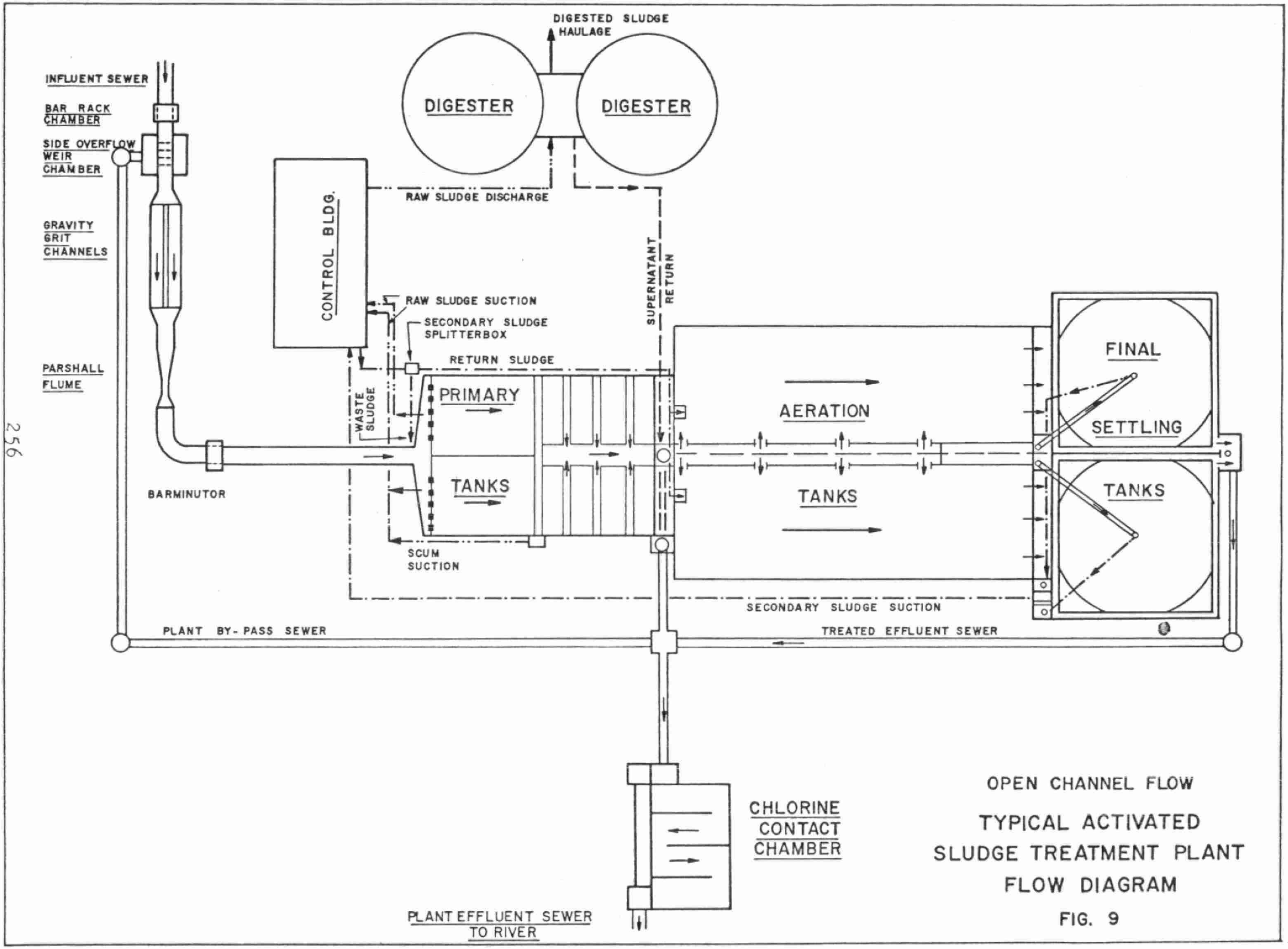


$$Q = C\,b\,H^{\frac{3}{2}}$$

Where C = coefficient = 2.85±

BROAD - CRESTED WEIR

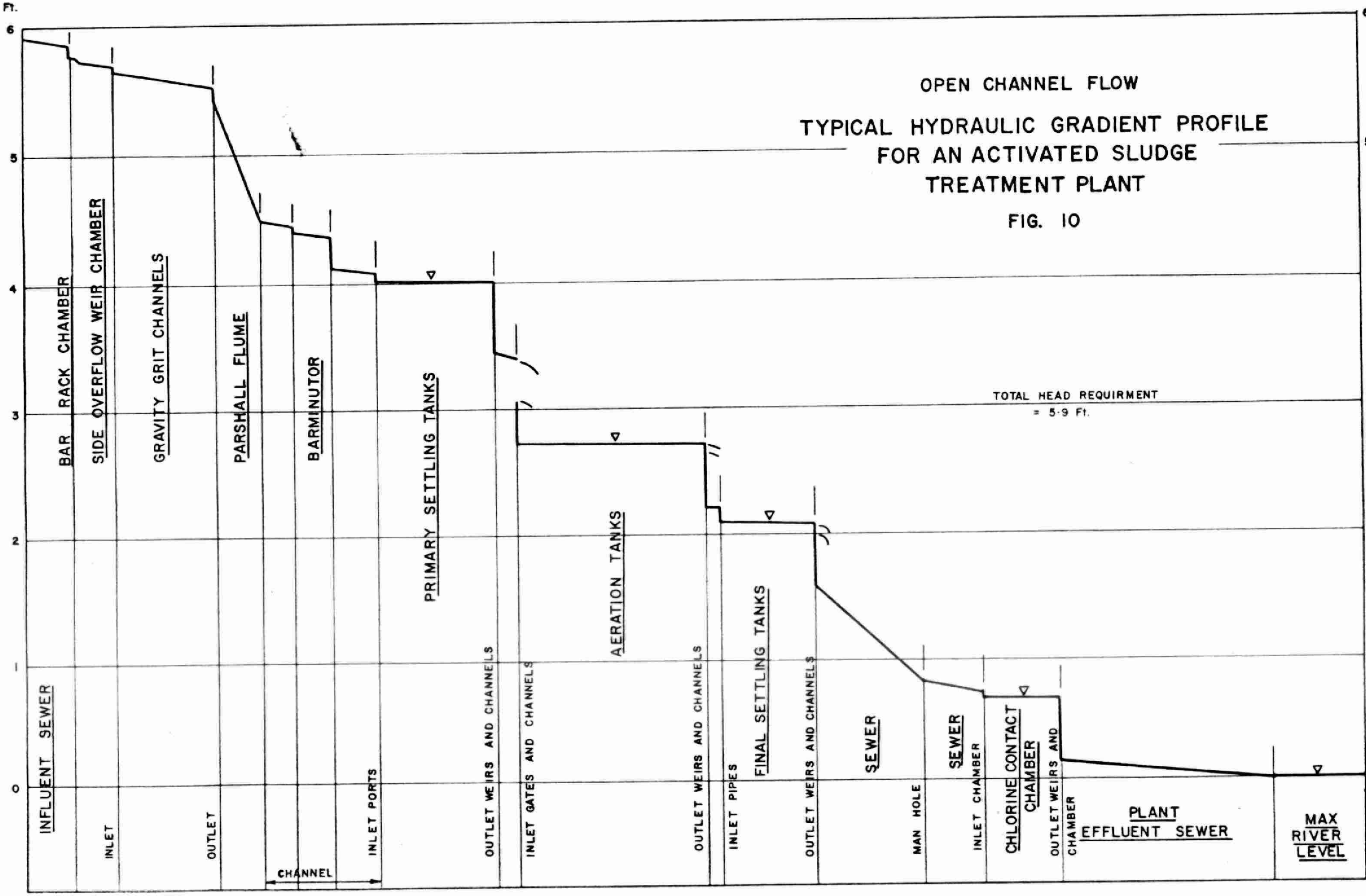
FIG. 6b



OPEN CHANNEL FLOW

TYPICAL ACTIVATED SLUDGE TREATMENT PLANT FLOW DIAGRAM

FIG. 9



OPEN CHANNEL FLOW

TYPICAL HYDRAULIC GRADIENT PROFILE FOR AN ACTIVATED SLUDGE TREATMENT PLANT

FIG. 10

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